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Flow Analysis of Hydraulic Connectors In Artificial Recharge System

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The Ohio State University
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FLOW ANALYSIS OF HYDRAULIC CONNECTORS IN ARTIFICIAL RECHARGE SYSTEMS A MODEL STUDY

by

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ABSTRACT

Artificial recharge techniques have long been used to augment ground-water supplies and increase the yield of infiltration galleries. A case history of the water-supply situation at Minot, North Dakota, which has long suffered from severe water shortages, indicates the usefulness of artificial recharge. Throughout much of the Minot aquifer, the water level was raised more than 20 feet within six months of artificial recharge operation. The cost of the system (about $193,000) was minute in comparison to an estimated cost of $12 million for a pipeline to the Missouri River.

A model study of the movement of a wetting front beneath an artificial recharge site, under both dry and damp conditions, illustrates the profound effect of permeability and stratification on the shape of the front.

A model study dealing with the shape of the cone of recharge in the vicinity of a recharge well shows that there is a sharp change in gradient directly under the well. Elsewhere the cone develops a shape that is generally consistent with theoretical studies. Where the cone of recharge crosses a boundary, indicating a change in permeability, the cone is slightly distorted.

INTRODUCTION

Artificial recharge is the practice of increasing the rate of infiltration, by diverting surface water or ground water into various types of structures that permit vertical movement of the recharge water into a ground-water reservoir. In most cases specific engineering techniques of this type are attempted in areas of substantial ground-water pumping where the water level has declined to such an extent that wells can no longer pump at their design capacities, there has been subsidence of the land surface, or water of undesirable quality has encroached into the aquifer. In a few other places artificial recharge has been used to control runoff and flooding by diverting excess flows into holding basins where the retained water may infiltrate. At still other places artificial recharge has been used to control the spread of contaminants in shallow aquifers.

In many areas throughout the world and particularly in arid and semi-arid regions, various industries and municipalities have been forced to depend on shallow ground-water reservoirs of limited areal extent for their water supplies. In cases such as these the transmitting capacity of the aquifer and the rate of natural recharge may be insufficient to meet the demands, but with low-cost diversion structures permitting the flow of surface water over the aquifer, the yield may be increased substantially.

Both direct and indirect means may be used to increase, artificially the rate of infiltration. Direct techniques include surface spreading of water by means of channel improvements, flooding, or the construction of canals, ditches, furrows or ponds. In addition, the injection of water through wells and shafts has been used successfully for decades. The dredging of the bottom of influent streams has also been widely employed in order to increase infiltration. This also occurs naturally in many channels following periods of high flows.

Artificial recharge techniques have been used for many years and particularly in Germany, Great Britain, France, and Sweden (DeFrance, 1894, Thiem, 1898, Richert, 1900, 1902, 1904, Gieseler, 1905, Scheelhaase, 1911, 1923, 1924, Riedel, 1934, and Soyer, 1947). In fact, approximately 12 percent of the water in West Germany is derived from artificially recharged ground water at the present time. More recently research and extensive construction has been carried out in Israel (See Symposium of Haifa by International Association of Scientific Hydrology, 1967).

Seepage of water from irrigation canals (a type of artificial recharge) in Fresno County, California was described as early as 1898 by Grunsky. Even earlier (1890) recharge ponds were constructed along the South Platte River near Denver, Colorado, in order to increase the yield of infiltration galleries.

A comprehensive literature search would probably reveal that artificial recharge techniques of one kind or another are used throughout the United States, especially in more recent years. In the greatest number of cases, however, only a limited amount of information has reached the general public. Perhaps the major reason for this aspect is that many experiments and actual facilities are completed with little publicity or fanfare by city engineers, plant managers, or ingenious individuals. The more widely known examples are those financed by various state or federal agencies, which incidentally are commonly very costly. Notable examples would include the extensive work presently being carried out in California, on Long Island and the extensive experimentation and operation in the Grand Prairie region of Arkansas.

Commonly artificial recharge facilities are constructed with little or no prior investigation or theoretical analysis. A great number of these work satisfactorily, but some probably don't work at all. In other cases, highly sophisticated systems are designed on the basis of extensive prior studies. A great number of these are adequate, but not uncommonly the costs of construction and pre-injection water treatment are so expensive that the facility is of questionable value. Nonetheless, in many cases, a great deal of money may be saved by using artificial recharge. An example illustrating this fact would be the City of Minot, North Dakota, the complete history of which is described in this report.

A major problem in the design of certain artificial recharge systems is the determination of the shape and development of the cone of recharge that forms in the vicinity of a well or pit. The shape of this cone will depend on several parameters, one of the most important of which is the thickness and permeability of rock units underlying the facility. Each change in vertical or horizontal permeability will tend to distort the cone. The shape of the cone, however, should dictate the lateral separation and the depth of recharge wells and pits.

The objectives of this report are to (1) describe the design, construction and operation of an existing facility, (2) to examine the infiltration of water through homogeneous and isotropic media as well as stratified material, and (3) to examine the shape of a cone of recharge in the vicinity of a well. The latter two objectives were based solely on model studies conducted in the laboratory. The experiments obviously do not represent actual field conditions, but they can be applied to them. There are, of course numerous problems involved in model studies.

There were three significant problems involved in these studies, one of which was technical. First, a great deal of difficulty was encountered in attempting to use a pressure transducer to measure the head in the model experiments. This was related to the extremely poor design and construction of the transducer system (a commercial item), which required extensive modifications and rebuilding. Secondly, the original manuscript was misplaced and thirdly, some of the equipment and records of the experiments were destroyed during campus riots in the spring of 1970.

Several undergraduate and graduate students participated in this project. The large laboratory model was designed and constructed largely by Mr. Ted Clark, an undergraduate. The small model used to examine the downward movement of a wetting front was constructed by Tom Schultz, also an undergraduate student who conducted the study. Much of the transducer work was carried out by Robert DesCamps, with the aid of Grahame Larson, both of whom were graduate students in the Department of Geology. Mr. James Edmonson continued the studies.

ARTIFICIAL RECHARGE AT MINOT, NORTH DAKOTA

A Case History

The history of the Minot's water supply represents what is probably a typical case that initially involved complacent attitudes on the part of municipal officials. This initial attitude was strengthened by inadequate advice and exploration. After years of water problems, administrators in Minot now are aware of the various parameters that must be considered during the various phases of water-supply planning and construction and are particularly aware of the value of artificial recharge of ground-water reservoirs.

Minot, first settled in 1886, is in the north-central part of North Dakota (fig. 1). The city, which supplies water to nearly 50,000 people, lies on the flood plain of the Souris River and the adjacent uplands. For more than a half century, Minot has had periodic severe water problems ranging from devastating floods to periods of no stream flow, and rapidly declining ground-water levels.

From its earliest days, Minot suffered from water-supply problems. The Souris River, which served as an early source of supply, would commonly cease to flow during prolonged droughts. This eventually led to the drilling of several test holes along the flood plain during 1915. In 1916, the first well, 132 feet deep, was drilled into the Minot aquifer (fig. 2). The success of this well showed that there was considerable potential for development of ground water within the valley. This led, in late 1917, to the construction of a second city well, also 132 feet deep. At that time, the city switched over entirely to the use of ground water. Reportedly, the depth to water in 1916 was about 2 feet below land surface or 8 to 10 feet above the bed of the

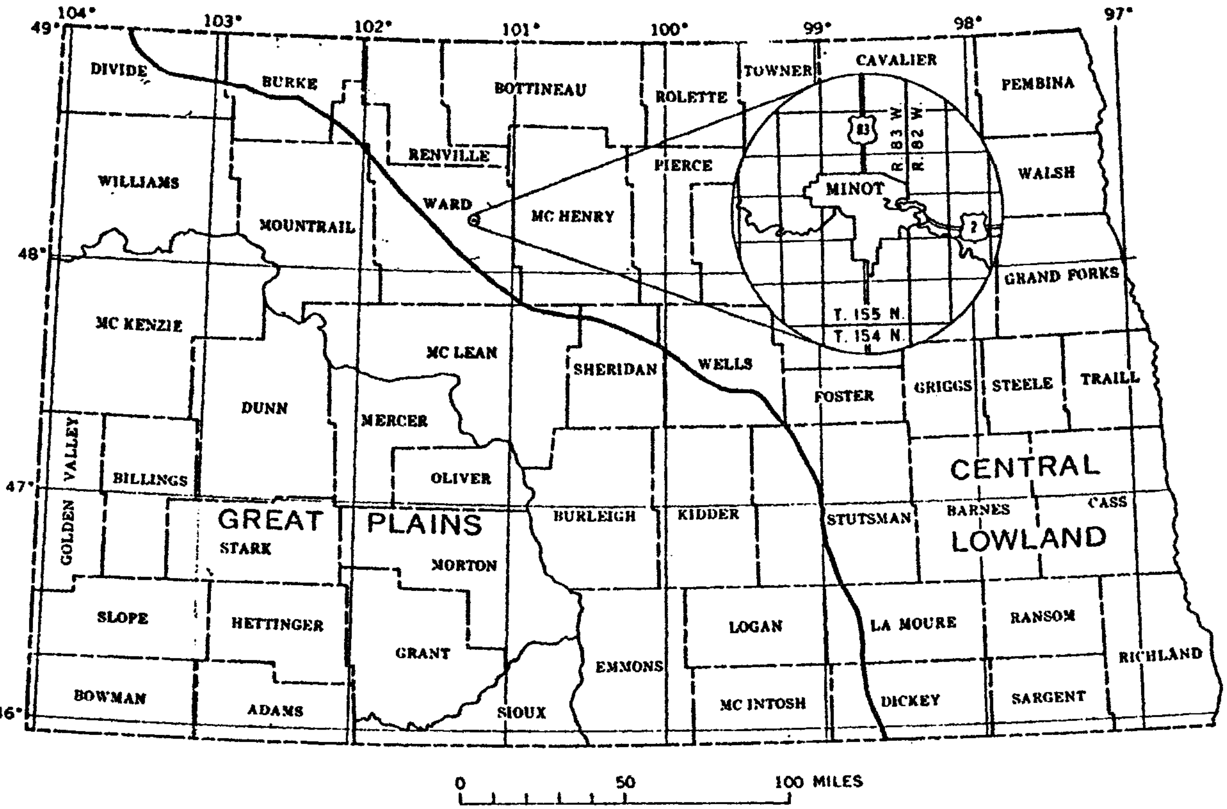

Figure 1. Physiographic provinces in North Dakota and location of Minot area.

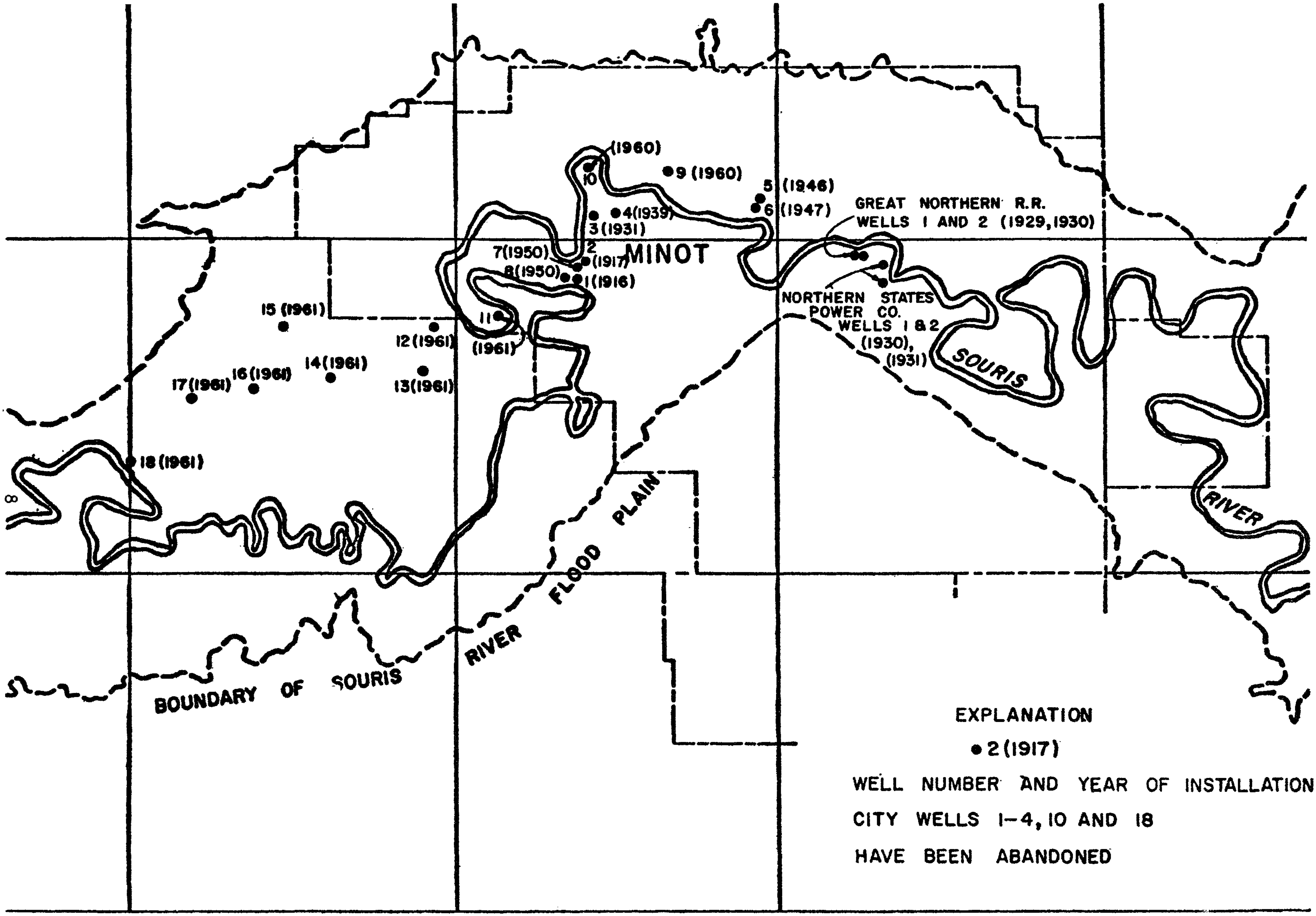

FIGURE 2 - Location of Municipal and Some Industrial Wells in Minot

Souris River (fig. 3). Thus, the aquifer not only was filled to overflowing, it contributed to the flow of the river.

In 1916-17, the average daily withdrawal from the two minicipal wells was about 600,000 gpd (gallons per day), and this increased to about 700,000 gpd by 1929. During the same interval, the ground-water level declined, amounting to some 4 feet during 1916-17, another 5 feet by mid-1918, and an additional 3 feet after 1918 for a total decline of 12 feet by 1929 (fig. 3).

The first industrial well tapping, the Minot aquifer, was drilled by the Great Northern Railroad in 1929; the railroad installed a second well in 1930. That same year a well, 100 feet deep, was also drilled by the Northern States Power Company. Because of increasing demand, coincident with prolonged drought, the city constructed its third well (158 feet deep) in 1931. The Northern States Power Company also completed their second well (109 feet deep). Pumpage from these 7 wells increased to about 1.4 mgd (million gallons per day) by the close of 1931 (fig. 2 and 3).

Beginning early in 1929, the water level began to decline rather rapidly - it declined an additional 14 feet between 1928 and 1931 to about 28 feet below land surface (fig. 3). Starting in mid-1932 demand steadily increased, and in 1939 city well 4, 154 feet deep, was drilled. At that time, pumpage slightly exceeded 2 mgd and the water level was about 48 feet below land surface in the vicinity of the wells. There was dewatering over a rather large area, accompanied by a decline in artesian pressure elsewhere.

In the years immediately preceding World War II, which was another dry period, the water level in the Minot aquifer declined to 58 feet below land surface by mid-1941 (fig. 3). The increased usage, 2.4 mgd, reflected rising industrial production, in addition to large quantities of water for

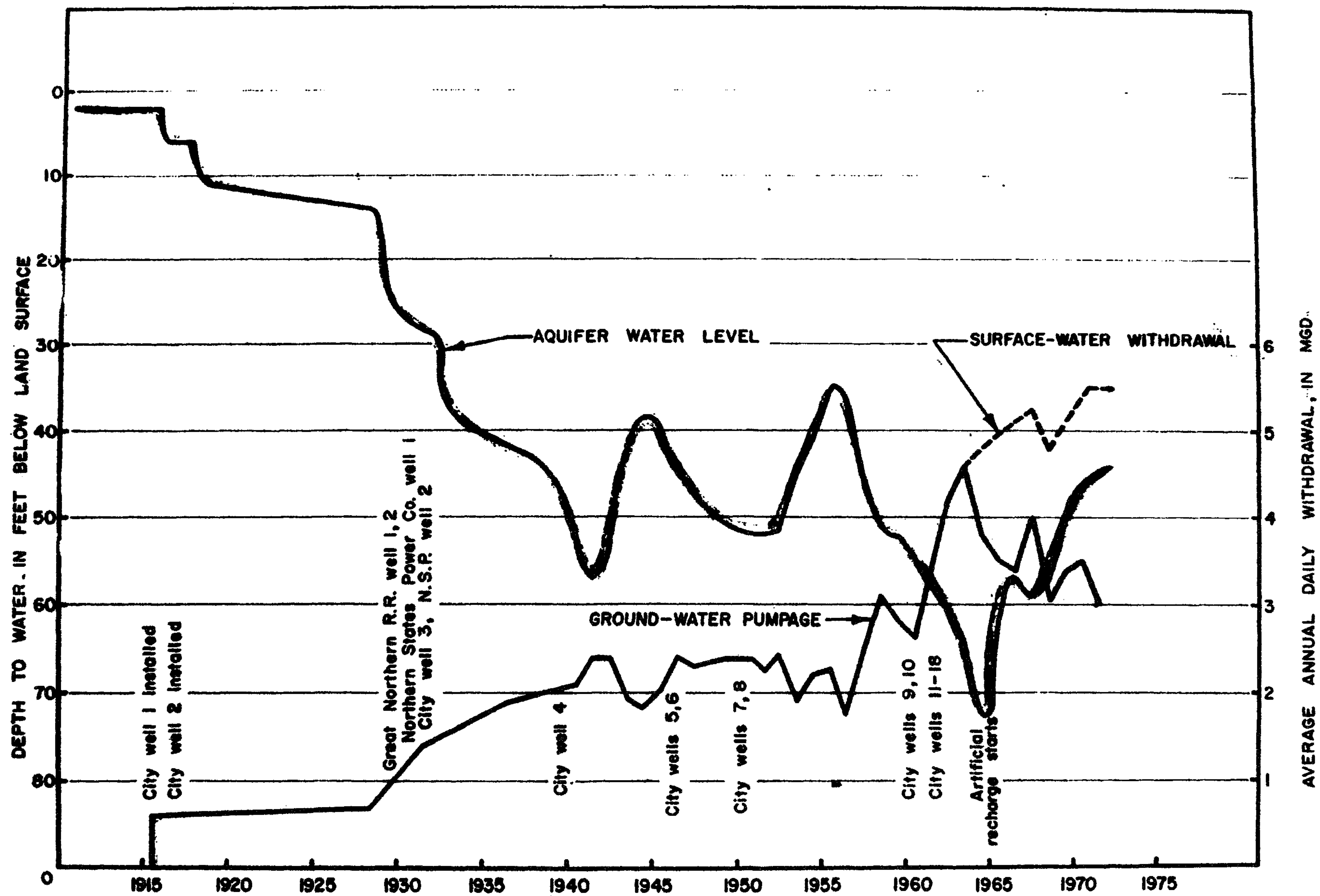

Figure 3. Relationship between pumping and water-level fluctuation between 1915 and 1972.

steam locomotives. City officials were again becoming concerned about a water shortage, but fortunately a wet period ensued, which provided considerable recharge to the aquifer. Coupled with a decrease in ground-water use in the mid-1940's, the increased precipitation and streamflow caused a striking recovery of the ground-water level. By 1944, pumpage had decreased about 25 percent to about 1.8 mgd, and the water level recovered to 38 feet below land surface--a rise of some 20 feet between 1941 and 1945 (fig. 3).

In 1943, the city council again voiced some alarm concerning the future of their water supply and well field, which resulted in 1944-46 in an investigation by the U. S. Geological Survey (Akin, 1947). Had the council been in possession of accurate water-level records, however, they would have known that the water shortage was by that time more apparent than real; the water crisis of 1941-42 had already passed, and the ground-water level had recovered to the position it held in 1934.

Following a comprehensive investigation, P. D. Akin, of the U. S. Geological Survey pointed out that there was no imminent shortage of water, and that if a shortage did occur in the future, artificial recharge might be a solution. The generalized depth to water, as it existed in March 1946, is shown in Figure 4.

Akin recognized that the sand and gravel forming the Minot aquifer was largely bounded by the valley walls. By analysis of aquifer-test data, he determined an average transmissivity of about 250,000 gpd/ft (gallons per day per foot) and a coefficient of storage of about 0.0003.

A conjectural statement in Akin's report, unfortunately taken out of context, was in future years to cause confusion and an unnecessary expenditure of money. Akin stated (1947, p. 3) that the aquifer's "linear extent is not known, but it probably extends many miles both upstream and

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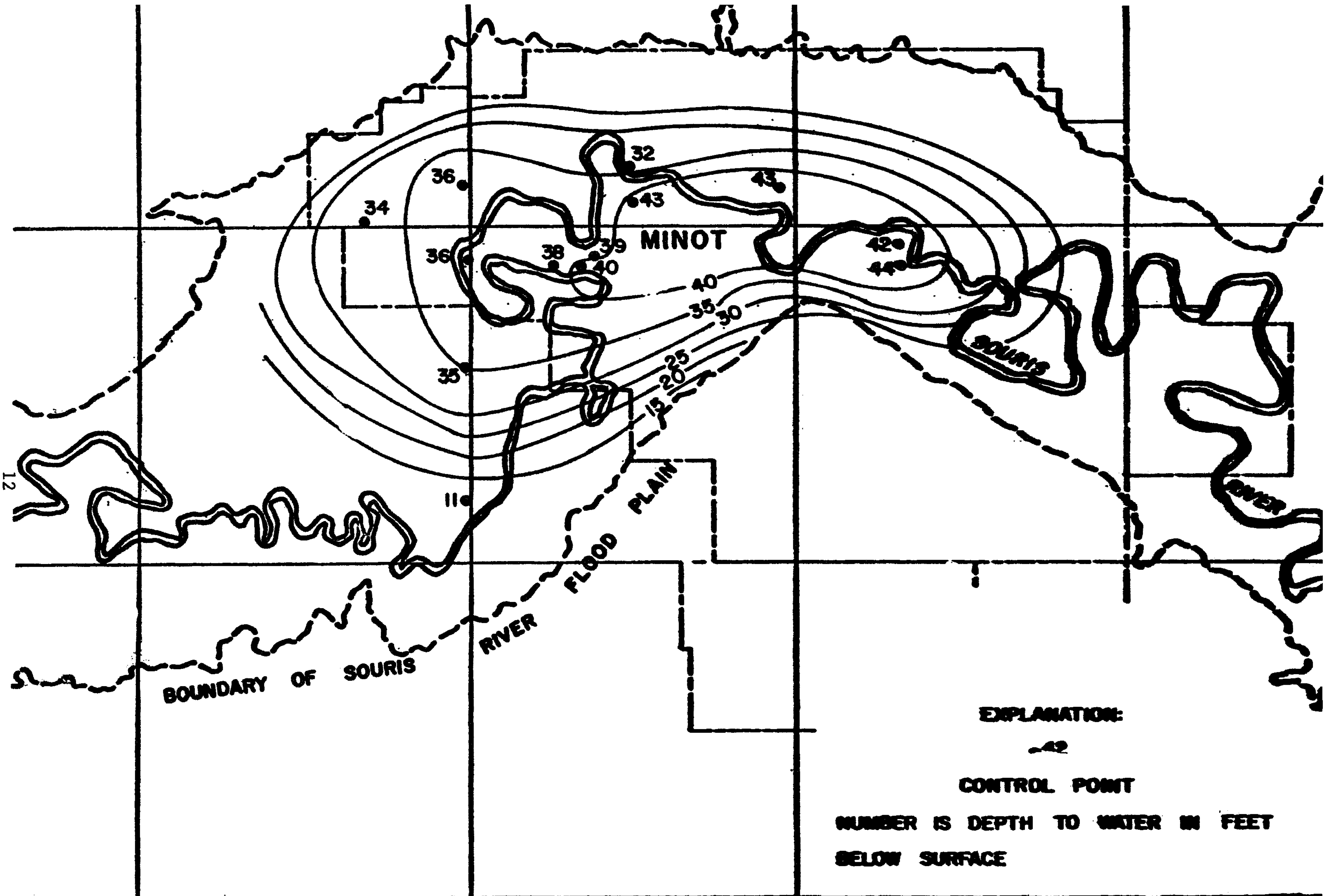

FIGURE 4 - Generalized Depth to Water in March 1946

downstream from the City of Minot."

Following Akin's investigation, the city drilled wells 5 and 6, about 150 feet deep, completing them in 1946-47 (fig. 2). Both wells penetrated a considerable thickness of permeable material, but unfortunately were drilled much too close together, being less than 30 feet apart. Although Akin had strongly suggested that records of water levels and pumpage be maintained on a regular basis, this was not done.

During the period 1945-1950, withdrawal rates remained nearly constant at around 2.4 mgd, but ground-water levels declined nearly 12 feet (fig. 3), due, in part, to a dry period. By 1950, water demand had increased enough to require the drilling of city wells 7 and 8. However, because the annual withdrawal rate did not increase substantially, water levels declined only one foot between 1950 and 1952.

Mid-1952 began a wet period that lasted until late summer 1955. Discharge of the Souris River increased substantially, and the stream channel was scoured enough to permit a large amount of infiltration. Throughout this time interval, the water level recovered some 17 feet (fig. 3). In fact, during the summer of 1955, the water level reached a level of about 35 feet below land surface, which was equivalent to its 1932 position. The trend was reversed during the next 5 years, however, and the water level had declined some 20 feet when city wells 9 and 10 were installed in 1960. Water use began to increase in 1956 from 1.7 mgd, reaching a peak of 3.1 in 1958, and then decreasing over the next two years to 2.6 mgd.

During the late 1950's, city officials decided to solve their water problems using one of two choices. Several believed that a pipeline to the Missouri River, some 47 miles to the south, would solve their problems indefinitely. An estimated 1959 cost for this facility was about $12 million.

The city could not afford such an expensive solution and officials again sought advice from the U. S. Geological Survey. Although there was no specific investigation, 23 test holes had been drilled during 1958-59 along four cross-sections of the river flood plain in a reach extending about 7 miles upstream from Minot. The logs of the wells revealed that the buried sand and gravel aquifer was very irregular in thickness and areal extent. Commonly consisting of very fine sand mixed with clay, the aquifer ranged in thickness from 0 to 88 feet, and averaged less than 20 feet. City officials had been informed, however, presumably on the basis of test-hole data and Akin's report, that they could produce 20 mgd, or 1 mgd per mile of river flood plain, in areas upstream from the city. The cost of such a ground-water system was substantially less than an alternate plan involving a pipeline to the Missouri River. A bond issue was passed, water rights to some 22 miles of flood plain were acquired and construction began on a major enlargement of the water treatment plan in anticipation of a bountiful and readily available ground-water supply.

Drilling of pilot holes for the new well field began early in 1961, but to the dismay and embarrassment of many, it soon became apparent that, contrary to reports, the aquifer did not extend for miles up the valley. This and subsequent testing proved that the aquifer was, for practical purposes, limited to an area within the corporation boundary. City officials had no choice but to rapidly re-evaluate the situation. So much money had already been invested in the project that their only choice was to drill additional wells within the city limits in the hopes that an adequate supply could be obtained. Within months, it became apparent that the new well field was far from adequate.

Water levels declined rapidly at the close of 1961 after completion of and subsequent heavy pumping from city wells 11-18. Pumpage exceeded 11 mgd on some days, but averaged about 4 mgd throughout the year. Periodic measurements of water-levels and pumpage began in January 1963. Almost immediately the continuing decline of water levels in the wells was noticed. The accelerated decline was largely due to drought conditions. The general decline continued into the winter of 1963, but a levelling trend is indicated on hydrographs from the early winter of 1963 through the fall of 1964 (fig. 5). This trend resulted from decreased pumping following the end of the drought in the summer of 1964. In any event, the long-term decline in water levels had amounted to about 5 feet per year from 1956 to 1964. By late summer, 1964, the non-pumping water level in the central part of the well field was some 70 feet below land surface and the upper part of a few well screens were probably being dewatered during pumping. The generalized depth to water, as it existed in July 1964, is shown in Figure 6. In the fall of 1963, the city of Minot was again facing a serious water shortage and the Assistant City Manager contacted the State Water Commission seeking aid and advice.

This shortly led to the second major geohydrologic investigation of the Minot aquifer, which began in the fall of 1963 (Pettyjohn, 1967). The investigation was a cooperative study in which the city, State Water Commission and the U. S. Geological Survey participated. Within a short time, it became apparent that the annual daily pumpage (4 mgd) from the aquifer exceeded the estimated rate of natural recharge (3 mgd) and that it would be of little value to deepen existing wells or drill new ones into the same aquifer.

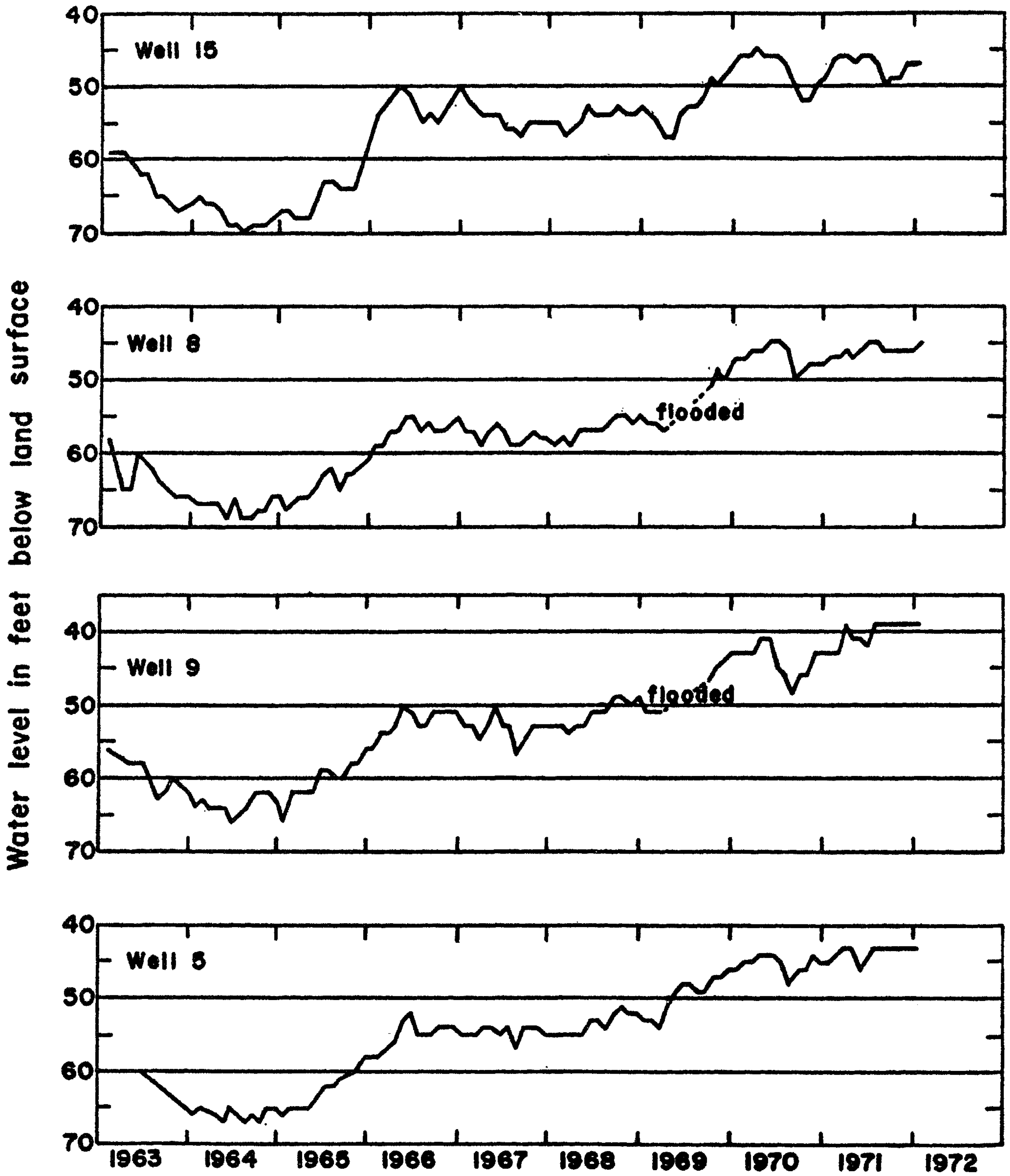

Figure 5. Fluctuation of water level in municipal wells 5, 9, 8 and 15 during the period 1963-1971.

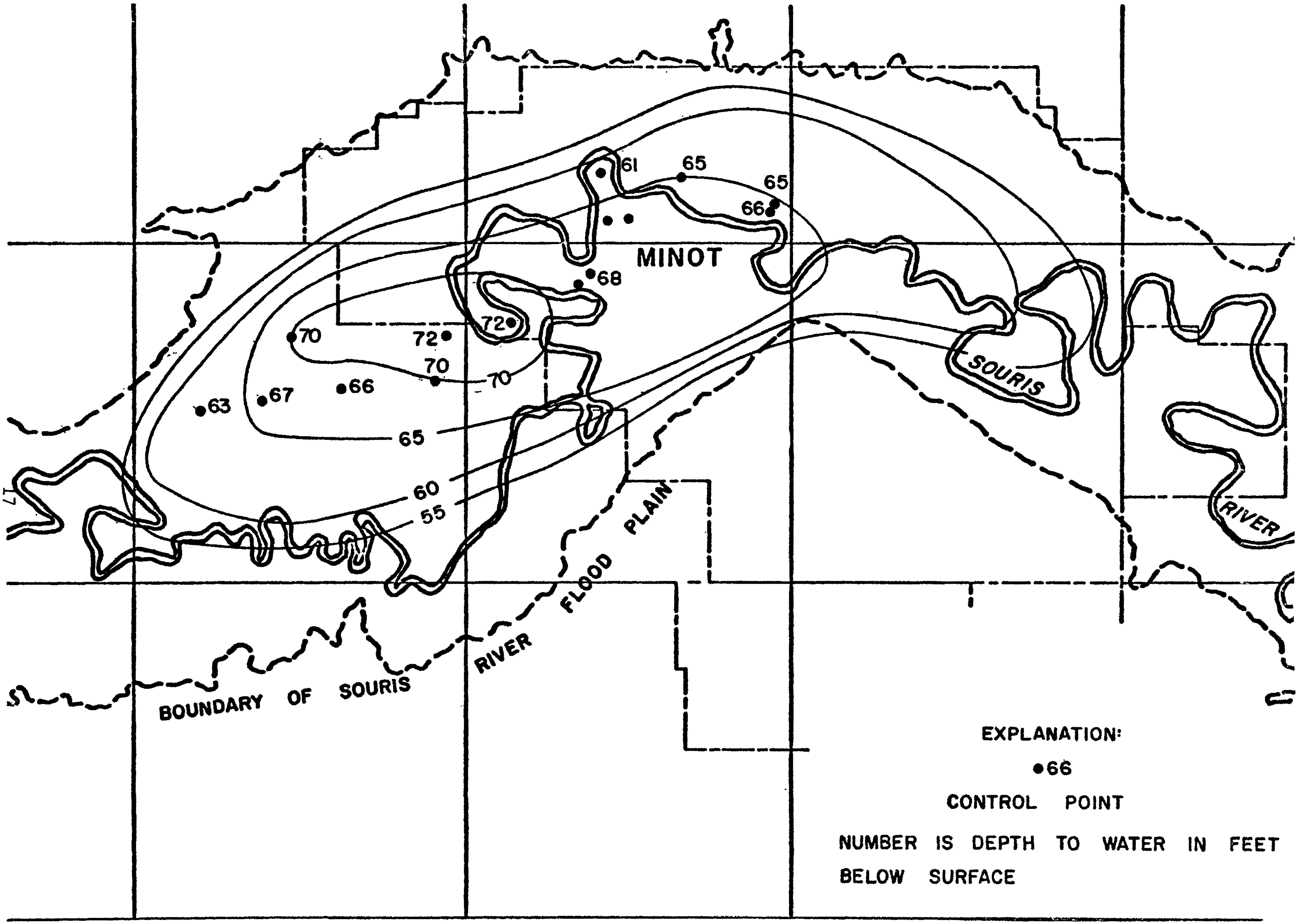

FIGURE 6 - Generalized Depth to Water in July 1964

Four solutions to Minot's existing problem were suggested, although not all were necessarily feasible; (1) construction of a dam on the Souris River flood plain to impound surface water, (2) construction of a pipeline to the Missouri River, (3) development of a well field in a thick but unexplored buried aquifer about 7 miles down valley and (4) artificial recharge of the aquifer to halt or reduce the water-level decline.

The first three possibilities exceeded both the time and financial resources available to the city. Consequently, it was decided to examine the feasibility of artificial recharge.

During the spring of 1964, part of an oxbow lake in a municipal park was excavated, through a shallow confining bed, to a depth of about 30 feet and used as a recharge pit. Subsequent tests indicated that it would be possible to recharge at the rate of about 1 mgd per acre of pit bottom, but owing to land acquisition problems, the concept of developing an artificial recharge site in the park was abandoned. The original pit was cleaned up, landscaped into a very attractive feature, and is still used for recharge although it isn't too effective due to the accumulation of mud and organic material on the bottom.

During the fall of 1964, investigation of the feasibility of recharging at another site began, and by winter, 1965, construction of a dual recharge system was initiated at the new site (Pettyjohn, 1968a, 1968b). The facility, located at the west end of Minot, is referred to as a dual recharge system because natural infiltration of surface water from a spreading basin through a surface layer of sandy clay is supplemented by flow through gravel-filled perforations in the clay layer, called "hydraulic connectors". The facility permits maximum water infiltration at nominal cost.

Preliminary recharge investigations began in an area of about 7.5 acres at the west end of Minot. The long, narrow wedgeshaped plot, which trends east-west, is about 1800 feet long and 260 feet wide at the west end (fig. 7). The property is bordered on the south by a railroad, on the west by a section-line road, and on the north by a housing development. Because of the size and location of the area, the land was of little economic value for other purposes.

Originally, two test holes, each about 65 feet deep, were drilled on or adjacent to the property and at opposite ends. Data from these holes indicated that the earth material at the site consisted of 7 to about 20 feet of sandy clay overlying 2 to 10 feet of sand, which in turn, was underlain by as much as 90 feet of sandy gravel. The water level was about 65 feet below land surface.

Eventually six holes, 30 inches in diameter, were bored at regular intervals along the northern boundary of the property. The holes averaged 25 feet in depth and as the casing was withdrawn, they were filled to within 5 feet of land surface with sewer rock. The object of these bored holes was to allow a determination of the rate of recharge at each site.

A hose was connected to a nearby fire hydrant with an inline water meter at the discharge end of the hose. Treated water was then discharged into each bored hole or hydraulic connector. Each hydraulic connector was tested at a constant head, approximately 20 feet total, and under variable head conditions. Rates of recharge at the six sites ranged from 30 to 90 gpm, averaging 60 gpm. The tests indicated that the underlying dewatered sand and gravel forming the upper part of the Minot aquifer had a considerable vertical and horizontal permeability. On the other hand, observation wells were not installed adjacent to any of the recharge wells thus making it

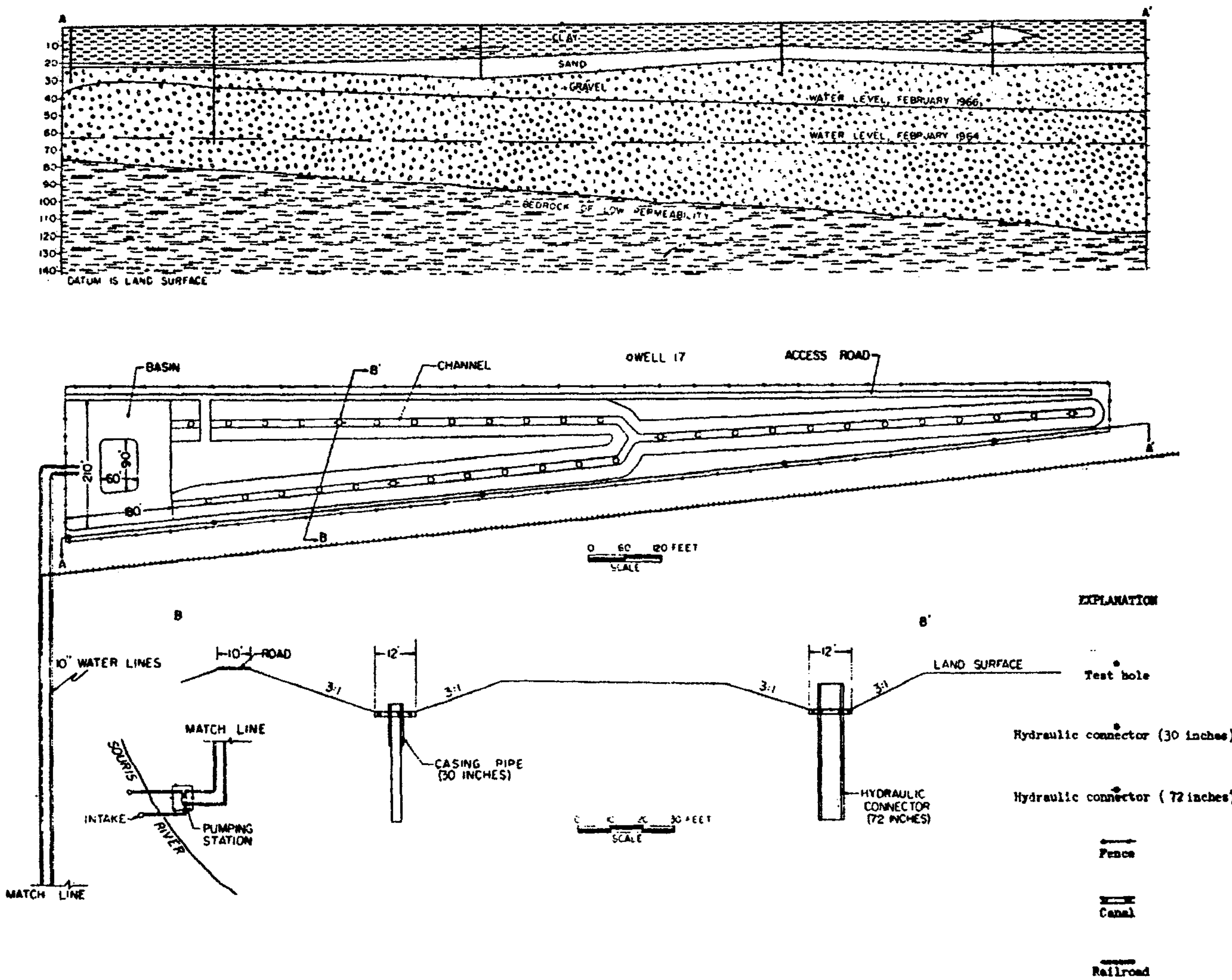

Figure 7. Geologic section, plan view, and cross section of Minot's dual artificial-recharge site.

impossible to determine the shape of the cone of recharge surrounding the wells. Had data of this type been subsequently available, it would have allowed a more feasible location of the recharge wells. Nonetheless, the experiments showed that the site had considerable potential for artificial recharge and, as a result, the city purchased the entire 7.5 acres. Since the recharge water would by necessity have to be pumped from the Souris River, the site had an additional attractive benefit since it was only about 1,000 feet from the river's edge.

A pit or holding basin was constructed at the west end of the site to provide a sediment basin since the raw river water is highly turbid. The dimensions of the pit at land surface are 180 feet by 210 feet; it is 35 feet deep. The upper part of the pit is constructed in clay, but the lower 15 to 23 feet were dug in unsaturated fine to coarse gravel. The pit was designed as a holding structure or sediment basin into which clay and silt would settle before the water was allowed to flow into the canals. Observations showed that initially the basin would recharge at least 2 mgd.

From the holding basin, a Y-shaped canal system was excavated in the overlying clay to a depth of about 10 feet with a bottom width of 12 feet and side slopes of 3 to 1 (fig. 7). It's function is to transport water from the sediment basin to the hydraulic connectors that are bored along the centerline through the bottom of the canals. Although some leakage (infiltration) occurs through the bottom and sides of the canal, the rates are too small to be effective.

The purpose of the hydraulic connectors, large-diameter perforations bored through the clay and backfilled with permeable material, is to provide high-permeability conduits through which the aquifer can be readily recharged with surface water from the canals (fig. 8). Originally, 36 connectors, 30

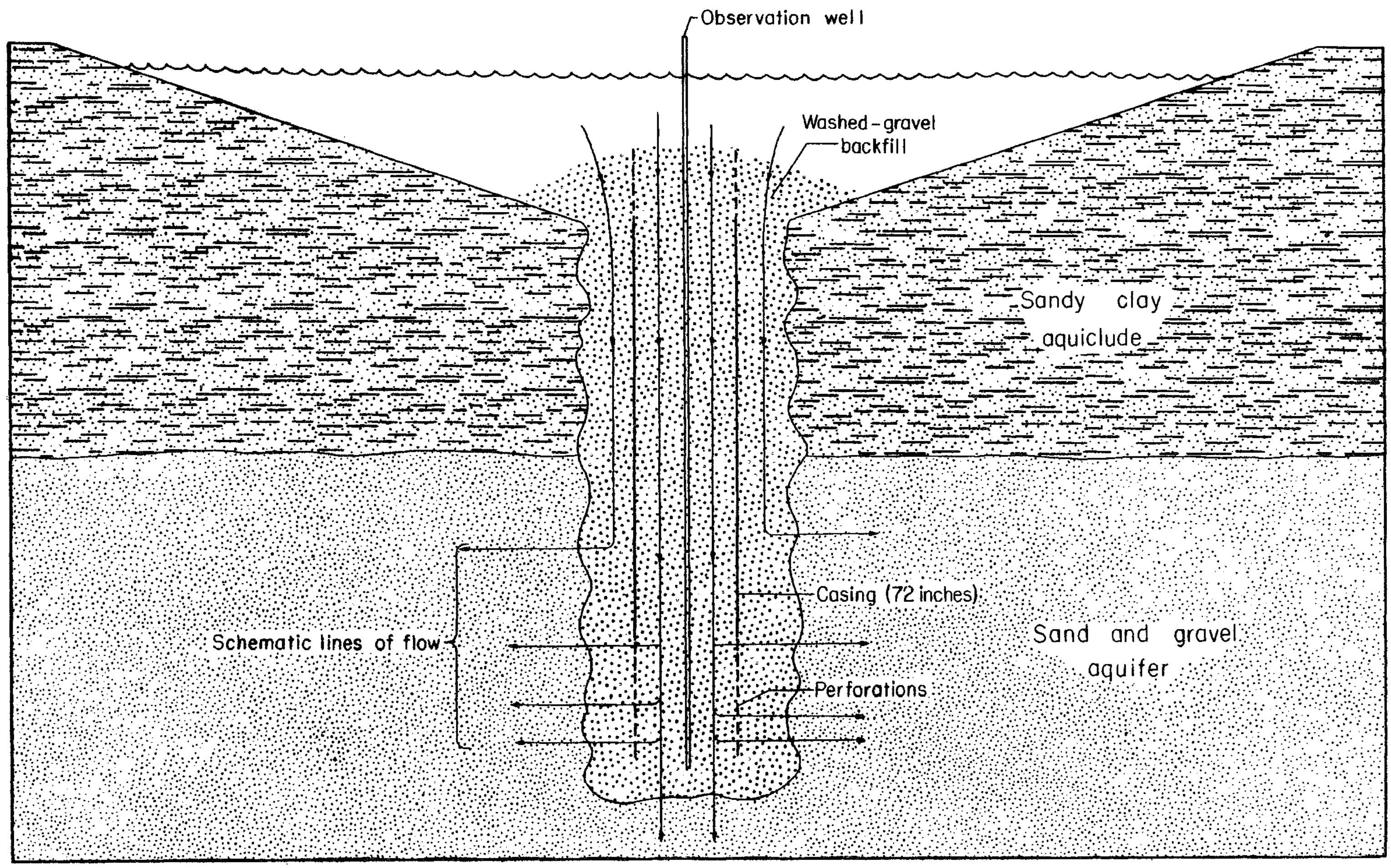

Figure 8. Detail of 72-inch diameter hydraulic connector.

inches in diameter, were bored along the canal centerline, ranging in depth from 28 to 32 feet.

Tests indicated that the hydraulic connectors have an average infiltration rate of 60 gpm (gallons per minute). It was assumed, therefore, that a total of about 3 mgd of river water could be recharged through the 36 connectors.

Several months after the recharge system had been in operation, it was found that the connectors, which were becoming plugged with clay, were almost impossible to clean. Consequently, four holes, approximately 12 feet in diameter, were excavated in the canals and 34 feet of 72-inch diameter corrugated culvert were installed in each hole. The lower 4 feet of each culvert were perforated to facilitate water movement. Each culvert was back-filled with washed 1/2-inch gravel both inside and outside of the culvert (fig. 8).

The water supply for the artificial-recharge system is obtained from the Souris River approximately 1,000 feet south of the recharge site. Two pumps were installed in an abandoned well house with intakes in the river. Because the abandoned deep well is connected to the city's system by a 10-inch cast iron main, it was relatively easy to supply water to the recharge site. A second 10-inch main was also laid. The installation resulted in a pumping rate of approximately 4 mgd to the recharge site.

A dam on the Souris River was constructed several hundred yards downstream from the recharge site in order to deepen the water over the pump intakes, to augment the surface-water supply, and to increase natural ground-water recharge in the vicinity of the well field.

The costs involved in the construction of Minot's artificial-recharge facility were small in comparison to the benefits received, insignificant if compared to the estimated cost of $12 million for the pipeline to the Missouri River, and infinitesimal if compared with the cost of a surface-water reservoir with a storage capacity equal to that of the Minot aquifer. The actual costs of constructing the entire recharge facility are summarized in Table 1.

Several major benefits have occurred through Minot's artificial-recharge operation. Of prime importance is the rapid rise in water level throughout the entire aquifer. This rise, in places, exceeded 20 feet within 6 months of operation. The generalized depth to water, as it existed in July 1971, is shown in Figure 9. The effect of recharging over nearly a seven-year period is shown in Figures 5 and 10.

Future municipal withdrawals can be increased because of the large quantity of water added to underground storage. During optimum operating conditions, at least 4 mgd are added to storage by artificial means, and at least 3 mgd by natural infiltration from the surface and from underflow from adjoining ground-water sources. The city, at present, withdraws an annual average of 2.4 mgd from wells, thus the net quantity of water than can be added to underground storage is about 4.6 mgd. Much of this water previously flowed unused down the Souris River.

The experiments, construction and subsequent operation of Minot's artificial recharge system proves that the concept, under proper hydrologic conditions, can be of significant value in the solution of water-supply problems. The techniques are relatively inexpensive, simple, and require only readily available equipment to construct and maintain. On the other hand, adequate testing is required to provide realistic and economically feasible designs.

TABLE 1.

Costs Involved in the Construction of the Minot, North Dakota, Artificial Recharge Facility

Recharge Site

Purchase of land	$ 8,228.00
Excavation of basin and canal system (87¢ per cubic yard)	35,060.35
Boring of 36 30-inch diameter and 4 72-inch diameter hydraulic connectors, including culvert	12,347.50
Site improvements (sod, plastic liner, security fencing)	16,885.00
Water transmission system (1,000 feet of 10-inch cast iron main, installation of pumps)	33,653.00
Souris River dam (city cost $57,000)	87,000.00
Total Cost	$193,173.85
Annual Maintenance and cleaning costs of entire facility	1,200.00
Estimate of costs for alternate method Pipeline to Garrison Reservoir (1959 estimate)	$12,000,000.00

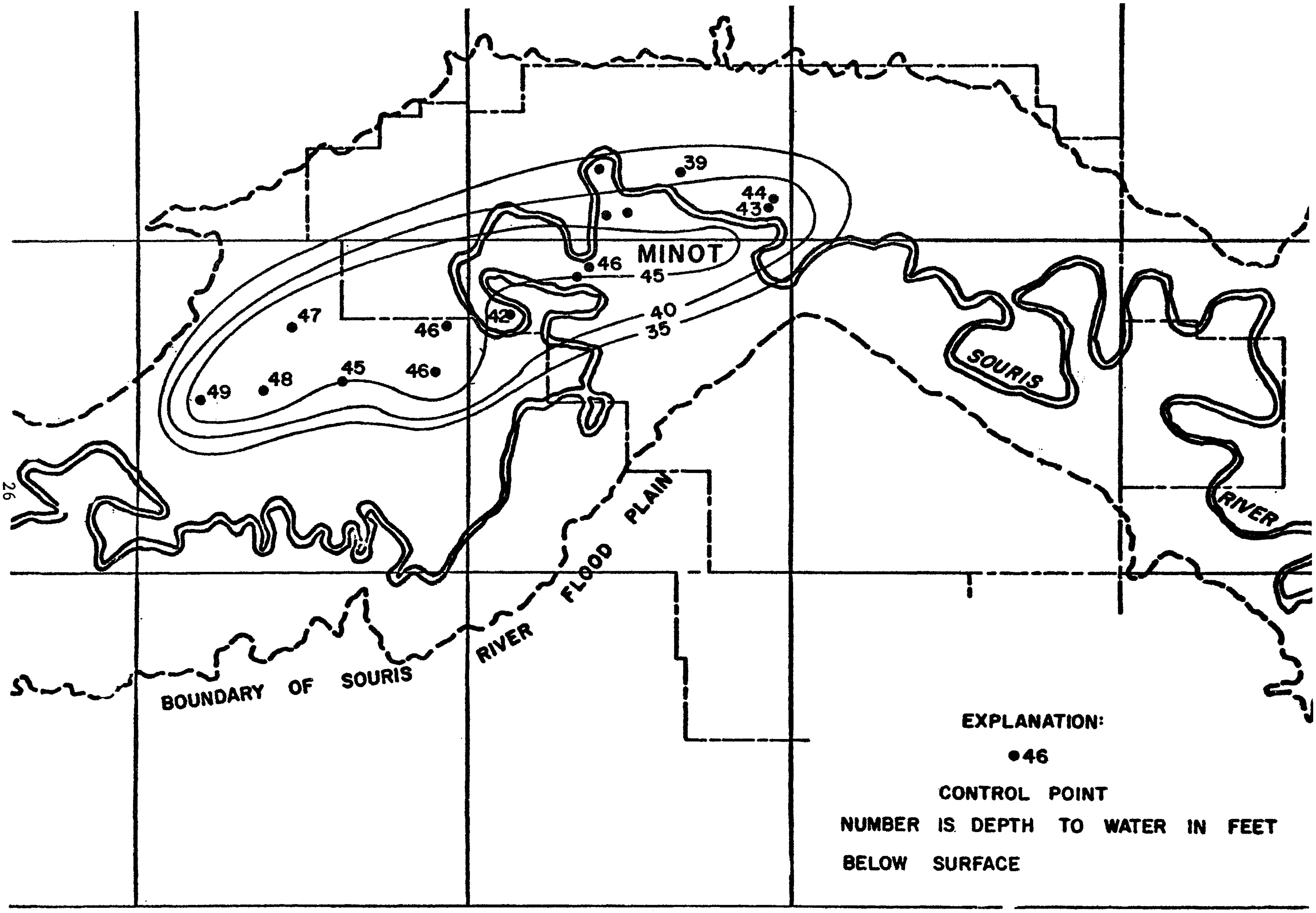

FIGURE 9 - Generalized Depth to Water in July 1971

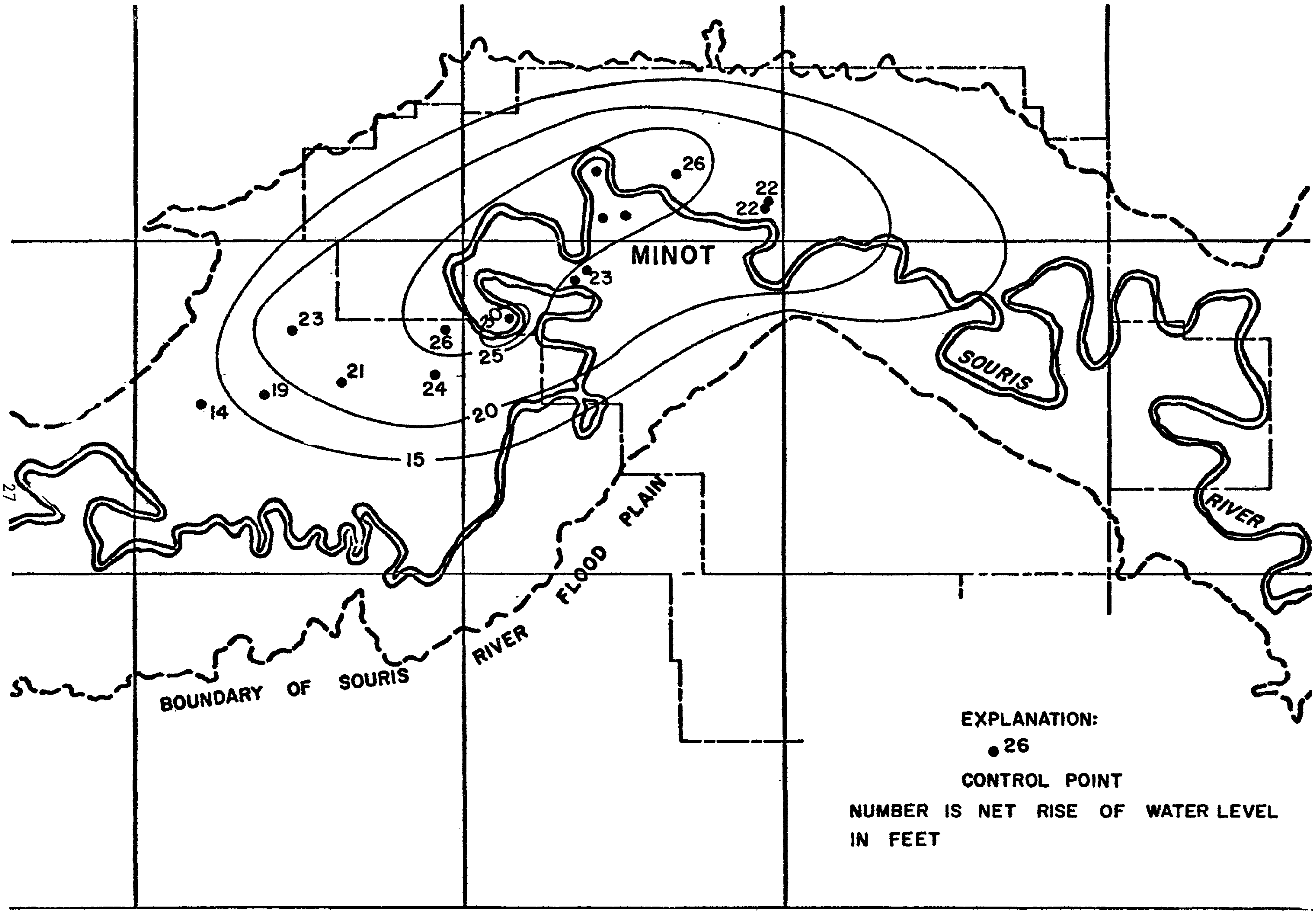

FIGURE 10 - Generalized Net Rise in Water Level, July 1964 to July 1971

MODEL STUDY OF THE MOVEMENT OF A WETTING FRONT THROUGH CLASTIC MATERIAL

In order to evaluate the movement of a wetting front beneath an artificial recharge system under various conditions, several model studies were conducted. This section describes one of those studies.

The primary objective of this phase of the study was to determine some of the qualitative properties of water infiltrating through a permeable unit following injection into a recharge pit. The properties could then be applied directly to the field situation, providing the field situation has been studied in sufficient detail to determine geohydrologic boundaries and properties. The tests dealt only with clastic material and is analogous to glacial deposits or other unconsolidated material encountered in the field. Even under the ideal, controlled conditions of laboratory testing, patterns of movement of the wetting front as it moves downward in unsaturated material can be studied and detected. It would also be possible to make quantitative measurements that could be correlated with actual field recharge conditions, provided the material was very similar.

Much work and numerous reports describe the actual field conditions under which ground-water recharge facilities have been designed and operated. Research in the laboratory, which simulates field situations, has been rather limited. The majority of this research has been under the direction of A. I. Johnson at the U. S. Geological Survey Hydrologic Laboratory in Denver, Colorado (Johnson, 1963).

Some preliminary library research into previous work on recharge models was conducted. With the facts gained from this research kept in mind, a model was designed and built. This work, however, parallels in some respect previous investigations by other researchers. With the aid of the theory of ground-water movement, the experimental results (figs. 13 - 20). were described and some meaningful characteristics derived.

The physical forces and their interactions, as they affect ground water, can be found in any elementary text of hydrology. The following forces, however, exert the main controlling effects on water movement. Gravity is the predominate force and it tends to pull mass upon the surface of the earth towards its center. The cohesion of the water molecules is the second important force and the adhesion of the water molecules with the surrounding rock particles is the third force. Finally, the rock characteristics (grain size, shape, sorting and distribution, porosity, specific retention, specific yield, and permeability) determine the final effect of gravity, cohesion and adhesion.

Most rock contains interstices, or void spaces. The space commonly is described quantitatively by a property known as porosity. Porosity is defined as the ratio, usually expressed as a percentage, of the volume of voids of a given rock mass to the total volume of the rock mass. For all practical purposes, ground water fills all void spaces in the saturated zone. From the previous definition, therefore, it follows that porosity is a measure of the quantity of water contained per unit volume (Todd, 1959).

Not all water contained in the saturated zone can be removed from the rock by gravity drainage. Specific yield, a percentage, describes the quantity of water that will drain by gravity from a mass of rock over a given period of time. That part of the water retained by molecular attraction and surface tension in the void spaces of the rock is known as retained and is described as the specific retention, also a percentage. The specific yield plus the specific retention is equal to the porosity of the rock.

Permeability is a measure of the capacity of a material to transmit water under a hydraulic gradient. It may be determined in the laboratory by observing the rate of percolation of water through samples of known length and cross-sectional area under a known difference in head.

The basic law for flow of fluids through porous materials was established by Darcy, who demonstrated experimentally that the rate of flow of water was proportional to the hydraulic gradient. Darcy's law may be expressed as $Q = PIA$ in which Q is the quantity of water discharged in a unit of time, A is the total cross-sectional area through which the water percolates, I is the hydraulic gradient, and P is the coefficient of permeability of the material for water. Permeability is defined (Wenzel, 1942) as the rate of flow of water in gallons per day, through a cross-sectional area of one square foot under a hydraulic gradient of one foot per foot at a temperature of 60° F.

The model for the infiltration tests was constructed of acrylic plastic 1/2-inch thick. The inside dimensions of the model are two feet wide, four feet high, and three inches thick; with an inside volume of two cubic feet (fig. 11). The model was bonded together with a suitable solvent and reinforced along the bottom and sides with screws. Two inches above the bottom of the model is a 1/2-inch thick acrylic plastic plate that forms a false bottom (fig. 11). The plate contains 22 equally spaced drainage holes 1/4-inch in diameter. Each hole is covered by a square of fine-mesh brass screen to prevent loss of granular material during testing. The perforated plate, along with the bottom part of the tank, forms a drainage collection tank for water after it filters through the overlying material. Vertical supports within the collection tank prevent cave-in or slippage of the perforated plate.

A small reservoir tank was constructed of 1/4-inch acrylic plastic (fig. 12). The inside dimensions are 23 inches long, two inches wide, and

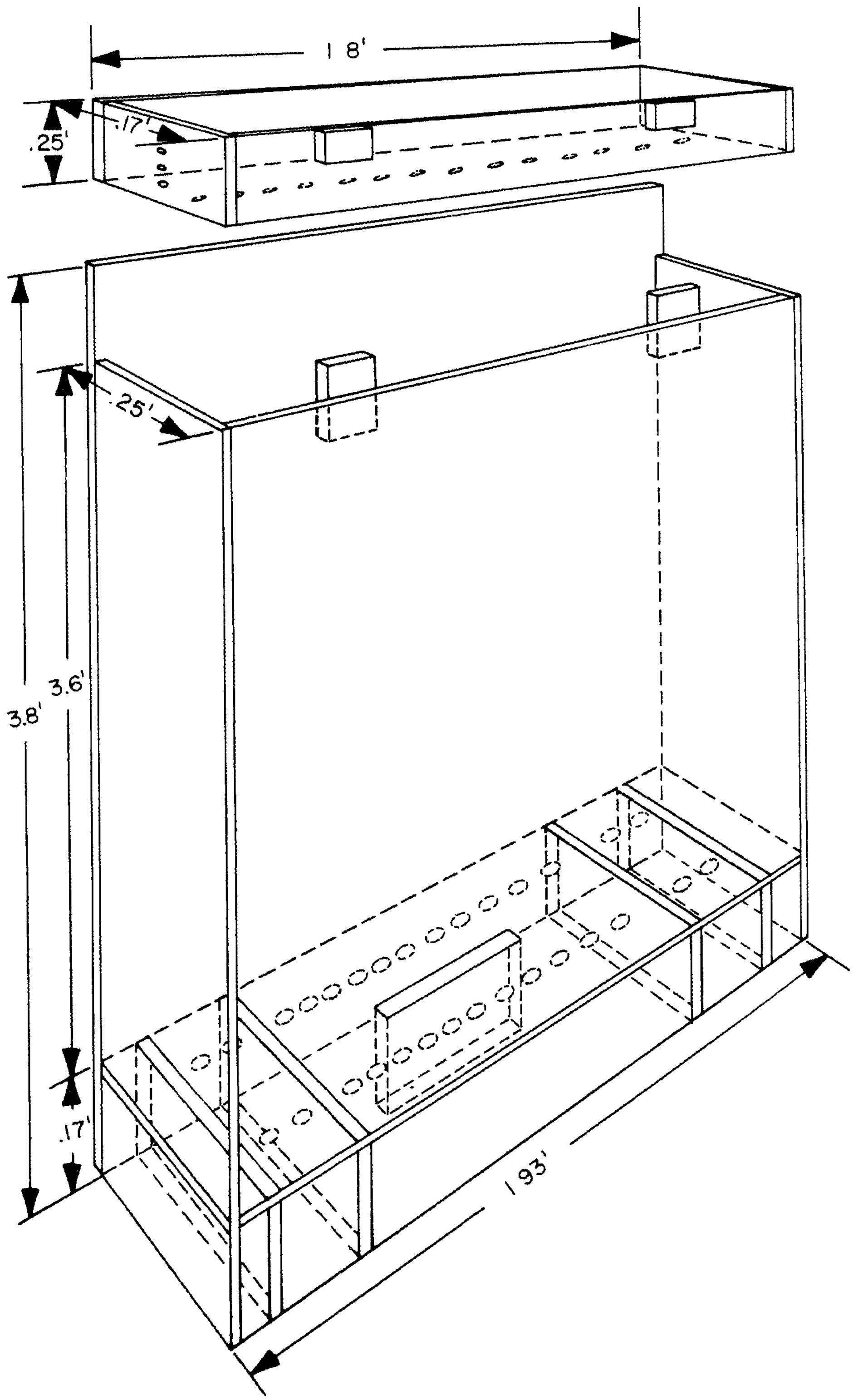

Figure 11. Model used in infiltration study.

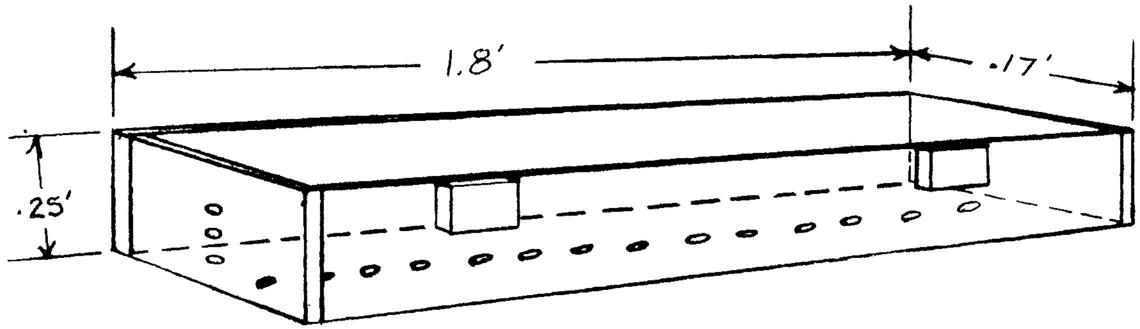

Figure 12. Reservoir tank of infiltration study.

four inches deep. The tank rests on the open top of the model. Four aluminum tubes project from one end of the reservoir tank. The tubes have an inside diameter of 5/32 inch and are spaced 3/4-inches apart with the bottom tube a quarter inch above the reservoir base. One of the tubes was used for inlet of water. Any of the remaining tubes could then be used for overflow of water. Consequently, the amount of head in the reservoir could be controlled by the choice of tubes. The minimum head allowable was 1/4-inch, whereas, the maximum head was 2-1/2-inches. Water dripped from the reservoir tank into the model through 1/16-inch diameter holes drilled along the bottom. Three holes are spaced equally along the reservoir bottom. Any number of holes could be drilled at any location on the bottom, if so desired. Holes not being used for a particular test were plugged.

The tests on the infiltration model system consisted of discharging a certain quantity of water per unit of time into the model, which was filled with clastic material. As water infiltrated, the movement of the front was marked at regular time intervals with a grease pencil on the tank front. The increments of time between successive markings varied between test runs to best depict the movement of the wetting front.

Two means of representation were used to depict the shape of the wetting front; photographs, and tracing on the tank front. The latter required a reduction to page size. Both methods have their advantages as well as disadvantages.

Tap water was used to maintain a constant head in the reservoir tank. The tap water became partially de-aired while in the reservior mainly because of the increase in water temperature. Although no measurements were made during these tests, de-aired water can have a marked effect on infiltration rate. Air bubbles in water can become entrapped in the pore spaces decreasing the permeability. Before each test, the discharge rate from the reservoir, per unit time, was measured. This and other data concerning each test run is contained in Table 2.

Infiltration tests consisted of measuring the movement and spread of the wetted front through four different configurations of clastic material: isotropic, heterogeneous, alternating layers of isotropic and heterogeneous materials, and stratified material. Each set up of the model was tested twice, once under dry conditions and once under damp (water of specific retention) conditions. Before each wet test was run, the sand-filled model was thoroughly saturated and then allowed to drain 10 to 12 hours prior to the test. The same procedure was carried out for all the wet tests. Each wet test was stopped once the wetting front reached the capillary fringe near the bottom of the model.

Five sets of tests were run. They ranged in complexity from an isotropic medium to a layered medium. By increasing the complexity in each successive set of tests, a particular characteristic could be studied, separately and then, in the next test, in conjunction with an additional characteristic. For example, a single infiltration site was used for the first set of tests, whereas, two infiltration sites were used for the second set.

The design of the reservoir simulated the conditions of an artificial recharge pit. Under actual conditions, a recharge pit would not be designed as the reservoir tank is in these tests. However, a "miniature" recharge pit was not used in the model because the rapid slumping of the clastic material.

Preliminary experiments showed that too high an infiltration rate confined the shape of the wetting front to a relatively narrow vertical column. For that reason, an infiltration rate of 15 milliliters per minute was used.

TABLE 2.

Specific Conditions During Model Tests 1-6

Test Number	Infiltration Rate	Marking Interval for Plate	Overflow	Material	Condition
1	15 ml/min	---	Second Tube	Isotropic	Dry
1	"	1 min	"	"	Wet
2	"	---	"	"	Dry
2	"	1 min	"	"	Wet
3	"	---	"	Hetero-geneous	Dry
3	"	10 min	"	"	Wet
4	"	20 min	"	Isotropic and Hetero-geneous	Dry
4	"	" "	"	"	Wet
5	"	" "	"	"	Dry
5	"	" "	"	"	Wet
6	"	6 min	"	Stratified	Dry
6	"	20 min	"	"	Wet

For the first set of tests, 3-1/2 feet of Ottawa Silica sand was placed in the model and mixed to achieve a uniform texture. The sand has an average size of .84 mm. The model very nearly simulated an isotropic aquifer. Some stratification was produced as the tank was filled with sand, but the effects were very minor, in both the dry and the wet isotropic tests.

Test One: The first set of tests served to point out the general shape of the wetting front during infiltration. An obvious difference could be noted between the wet and dry isotropic tests. In the dry test, water moved from the infiltration site more rapidly vertically than horizontally. In the wet test, the retained water in the sand afforded more lateral movement of the wetting front (fig. 13).

Each test on the model determined a characteristic that could be applied to an actual recharge pit. In the first series of tests, a comparison was made between the spread of the wetting front in the model as compared to situations where material underlying an artificial recharge area are nearly dry and when it has some retained water. If the material beneath the recharge pit is dry, the infiltration front will produce a narrow column of wetted material extending from the pit to the water table and apparently subsequent flow (infiltration) will occur along this zone. If the deposit is already wetted, or when it becomes so, the wetting front will spread laterally more than it will move vertically. Thus, in the recharge area, the spacing of pits will have to be strategically located so as to provide maximum efficiency for the available water. A period of time at the beginning of operations will be inefficient until the area between the pit and the water table becomes wetted. Pits should be spaced on the basis of how the system will operate after the area beneath the pit has been wetted and is flowing.

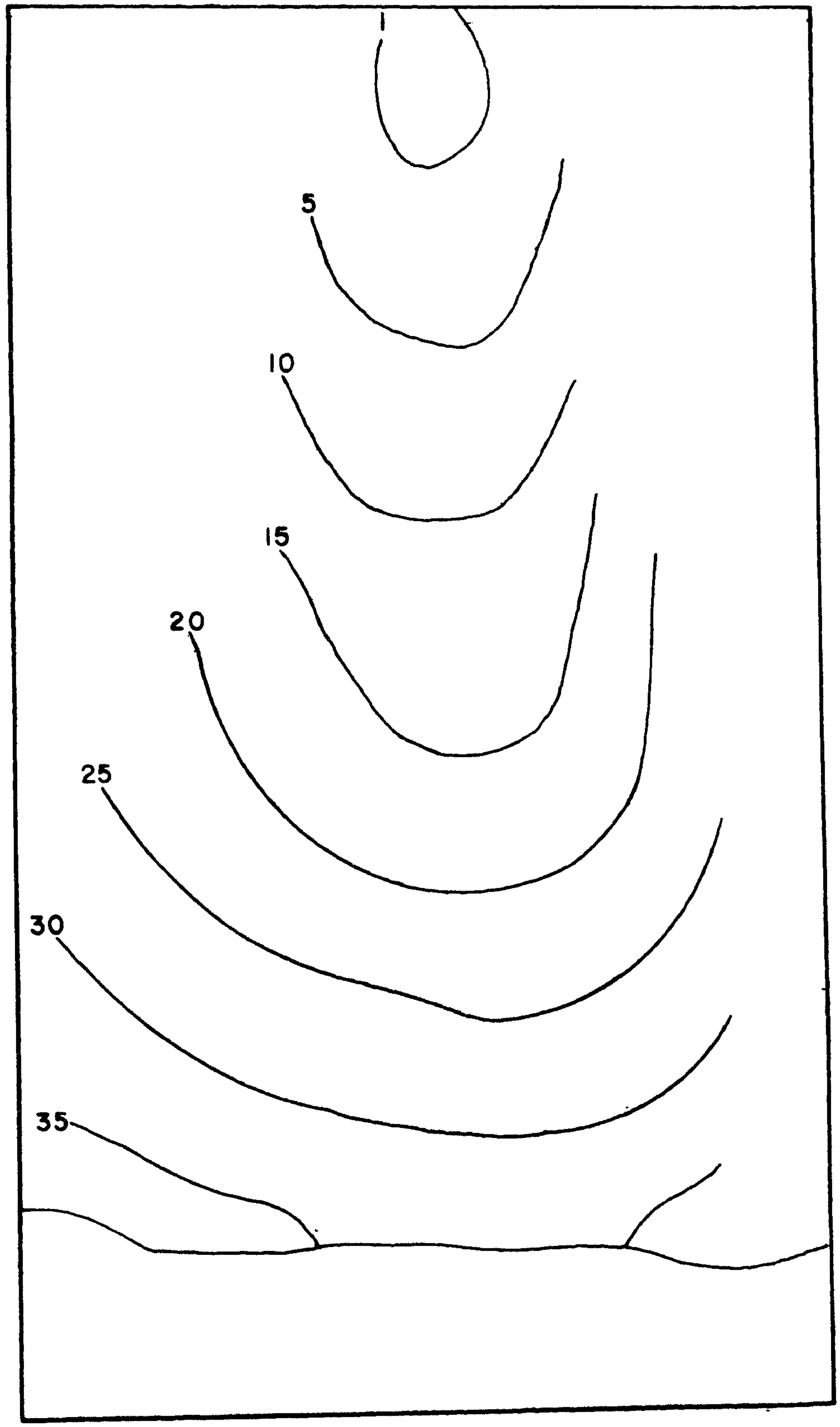

Figure 13. Movement of wetting front in damp material during test 1. Numbers are minutes.

Test Two: The purpose of the second set of tests was to study the effect of two infiltration sites on the movement of the wetting front. Again, the tests were conducted using Ottawa sand. As in the first tests, the wetting front in the dry material moved downward vertically more quickly than when the sand was wet. The wetting fronts from the two sites, in both dry and wet tests, had no effect on each other until they coincided about half way down in the model (fig. 14). At this point, the combined fronts began to spread laterally. Had the model been wider, the movement of the fronts could have been realized to a greater extent.

Consequently, in the second set of tests, optimal spacing between recharge pits was studied. It is important to denote here that as the material for some distance under and surrounding the pits becomes wetted, the volume of material wetted begins to reach a constant amount. To ascertain the correct spacing of pits, the depth to the water table seems to be most critical. Assume the flow to be nearly unobstructed by lenses of material of different permeability, then, as the depth of the water table increases, the pits should be spaced more closely because of less lateral spread. Obviously, the more pits the greater the volume of water that could be introduced underground. But, the cost of construction is a limiting factor. Therefore, efficient locations for the pits is of prime importance.

Test Three: The third set of tests consisted of tracing the wetted front as it infiltrated through a heterogeneous mixture of material ranging in size from fine sand (1/8-1/4 mm) to pebbles (5 mm). The sample material was collected from a gravel pit in the vicinity of Columbus, Ohio. The material was collected from several lenses of glacial outwash about 10 to 15 feet below the top of the pit. The material has an average permeability of 725 gpd per square foot.

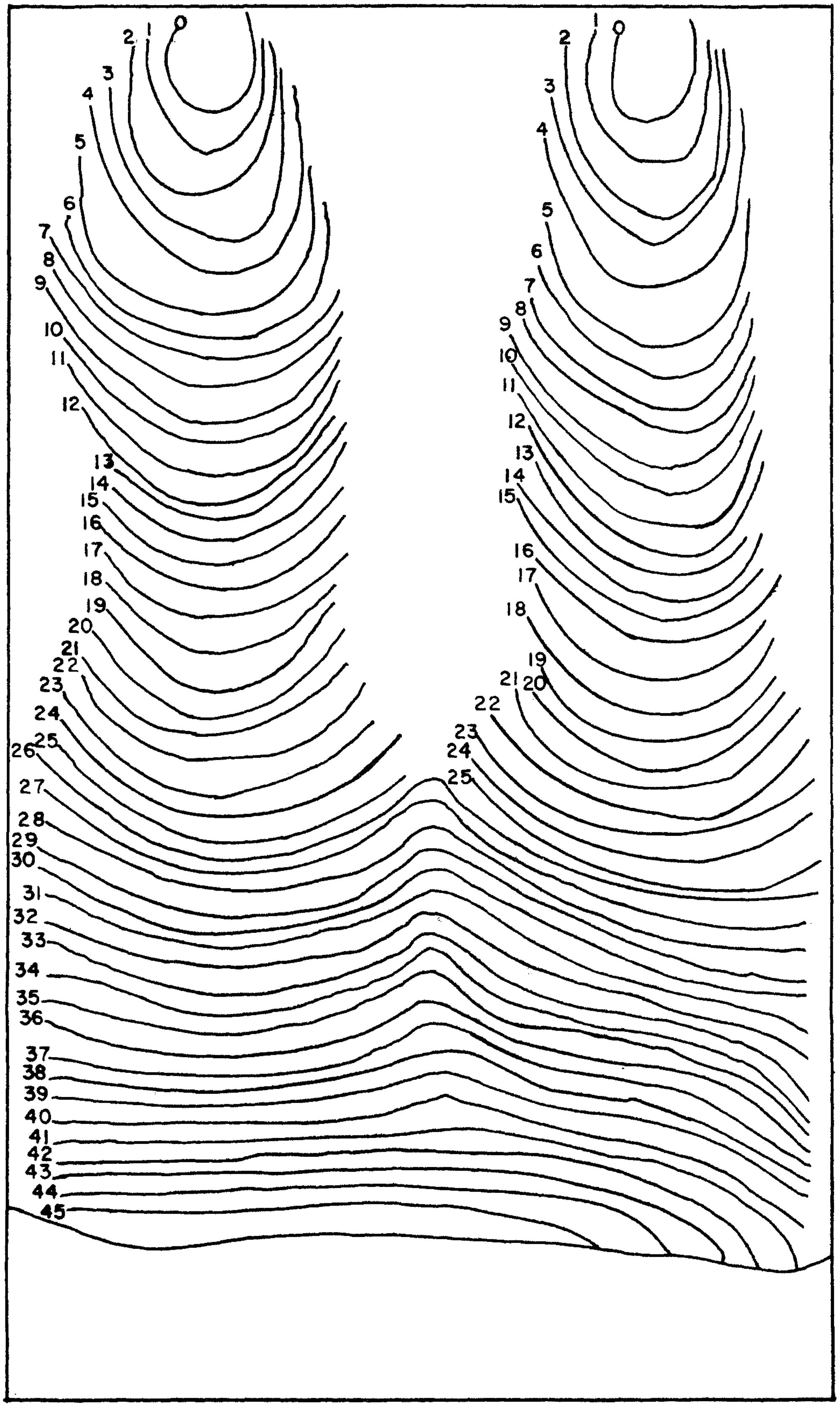

Figure 14. Movement of two wetting fronts in damp material. Numbers are minutes.

The tank was filled with 3-1/2 feet of unsorted and unstratified outwash material. Once again, the same relative differences were noted between the wet and dry tests. As expected, the lateral spread of the front, in both the wet and dry tests, was more pronounced than in any of the previous tests. The lateral spread can be attributed to increased capillary action due to the smaller size ranges of the sample. Figure 15 depicts the pattern of infiltration for the wet test, which had a more even pattern than the dry test.

As can be seen from the third set of tests, the size and distribution of material is of prime importance in the shape and movement of the wetted front. Consequently, in actual field tests, detailed sampling of the subsurface is required. The areal extent, stratigraphy, and thickness of the material above the water table must be mapped. Permeability tests should be run to determine the maximum rate of infiltration that will be possible with a certain head. It would be nearly useless, for example, to place the recharge pit in a clay pit relative to placing it in a sand deposit.

Tests Four and Five: The fourth and fifth sets of tests had 1-1/2 feet of Ottawa sand and 1-1/2 feet of the outwash material. In the fourth series of tests, the channel material was overlain by the Ottawa sand (figs. 16 and 17). In the fifth series of tests, the positions of the sample were reversed (figs. 18 and 19).

The shape of the wetting front in the dry test (fig. 16) in the fourth set of tests, reflects a significant change. The wetting front moved down normally through the isotope material until it reached the interface. At that point, the wetting front did not immediately cross the interface, but moved laterally. Eventually, the front crossed the interface and moved downward into the heterogeneous material much more rapidly than it had in the overlying sand. There was very little lateral movement of the front below the interface.

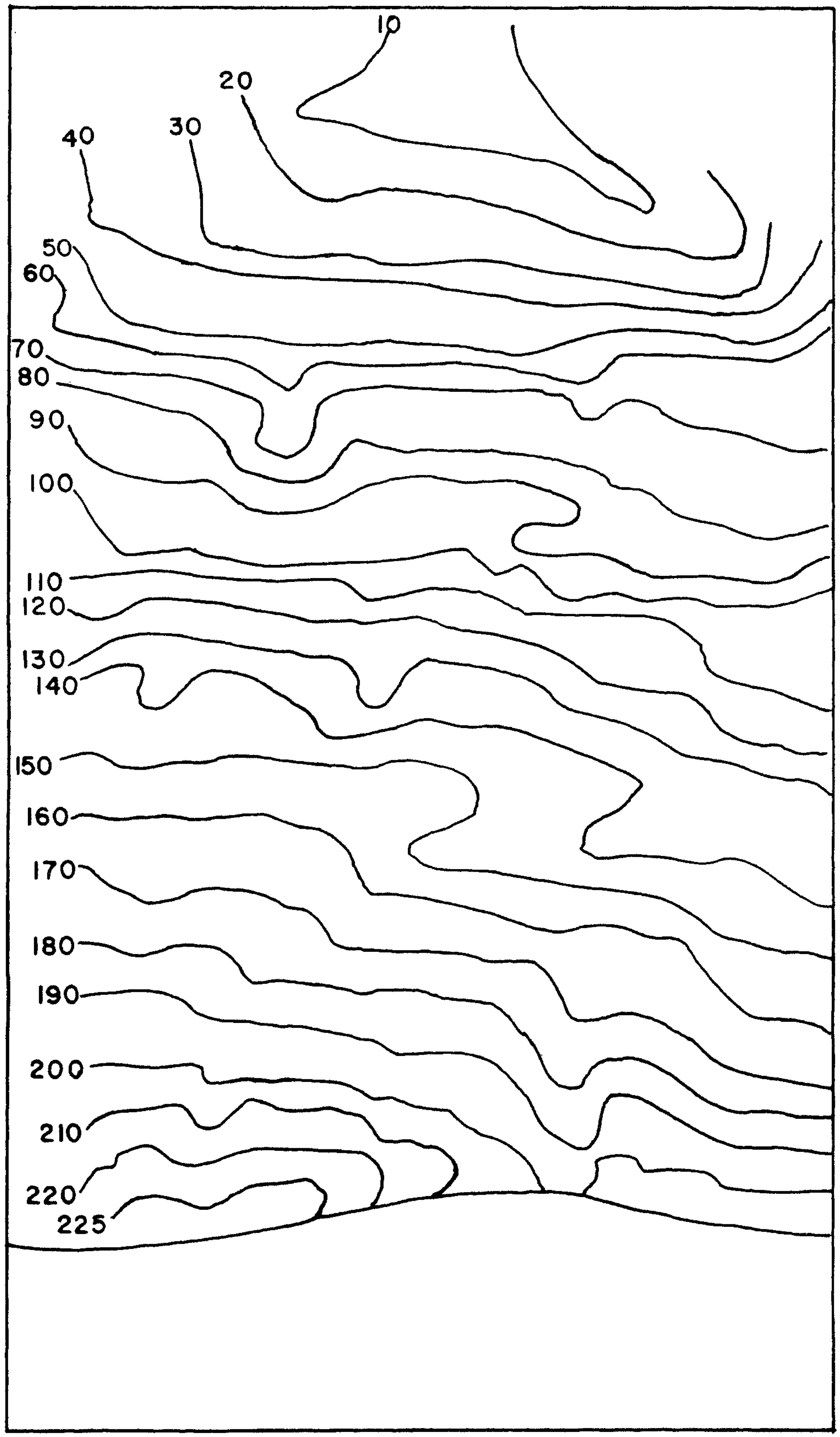

Figure 15. Movement of wetting front in damp heterogeneous material during test 3. Numbers are minutes.

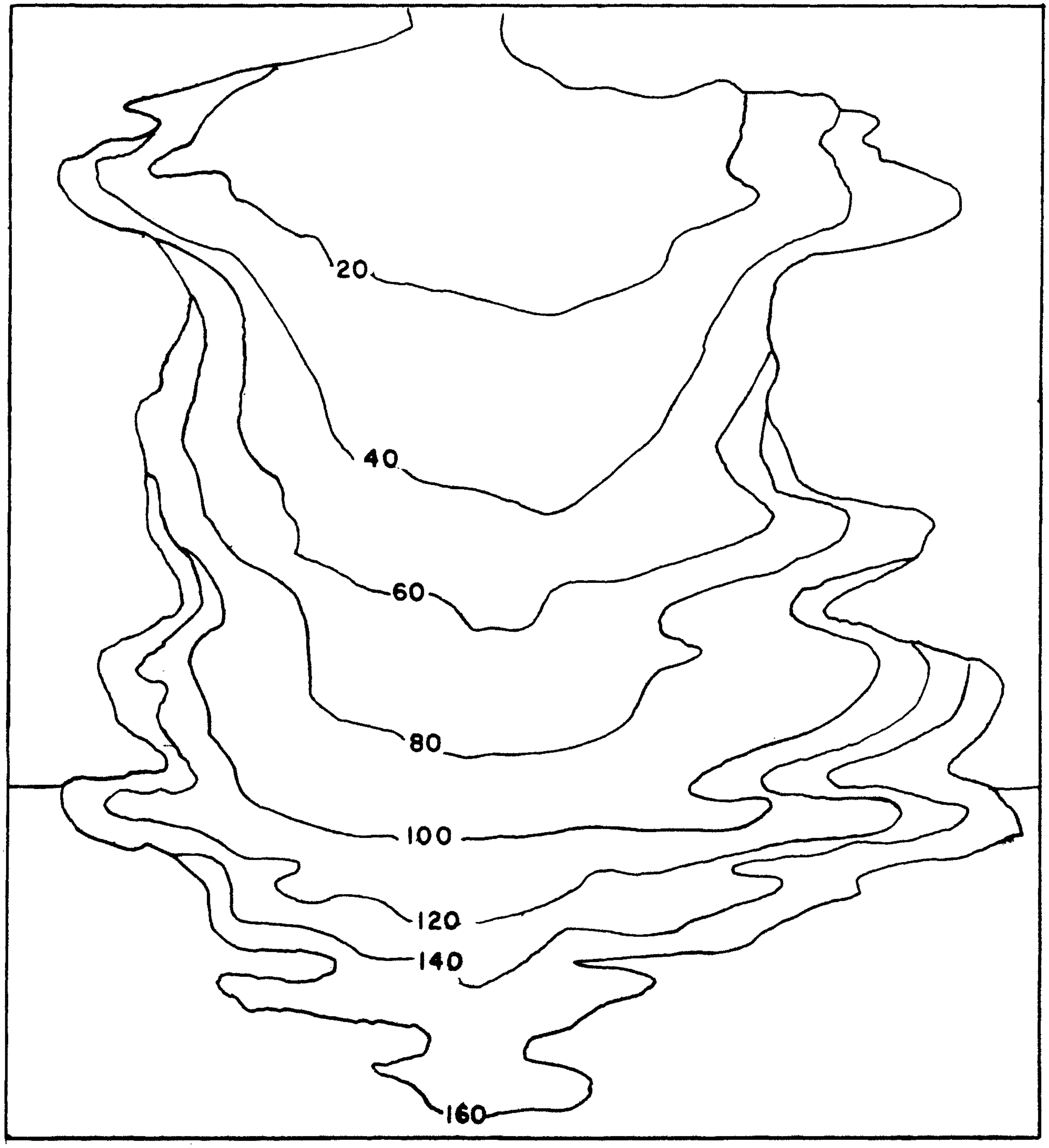

Figure 16. Movement of wetting front in dry material with one boundary. Upper material is Ottawa sand, lower sediment is outwash. Numbers are minutes.

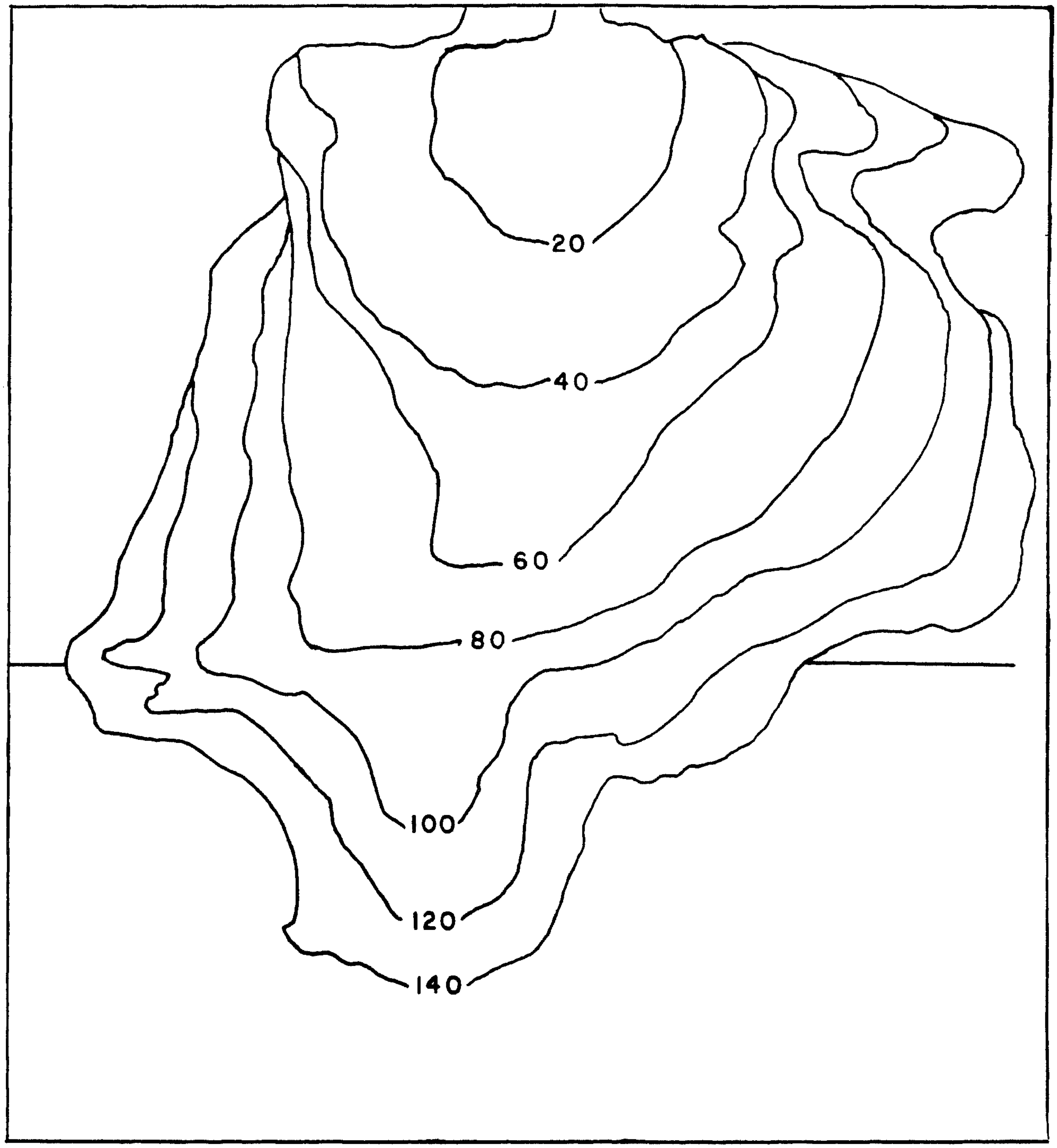

Figure 17. Movement of wetting front in damp material with one boundary. Upper material is Ottawa sand, lower sediment is outwash. Numbers are minutes.

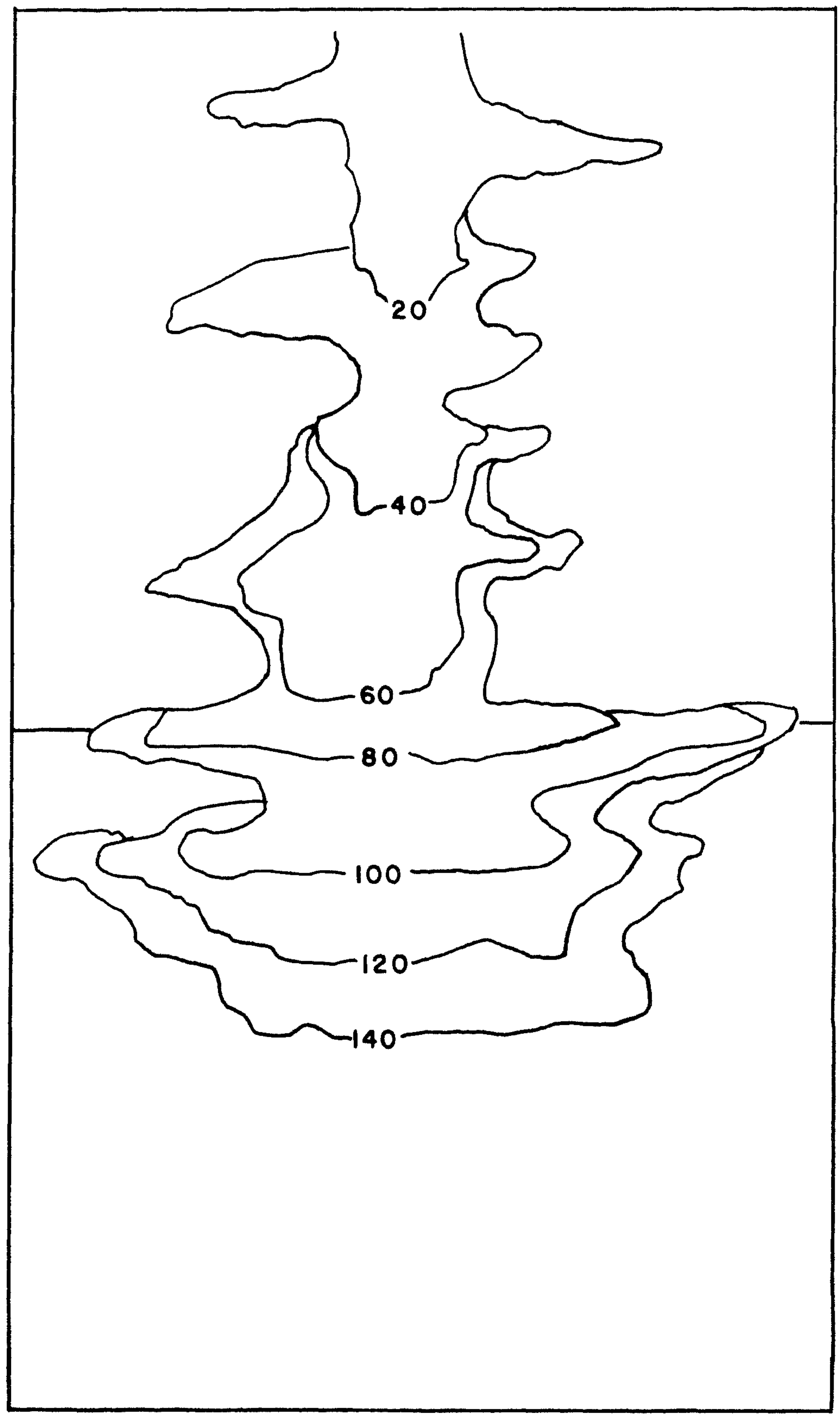

Figure 18. Movement of wetting front in dry material during test 5. Upper material is outwash, lower sediment is Ottawa sand. Numbers are minutes.

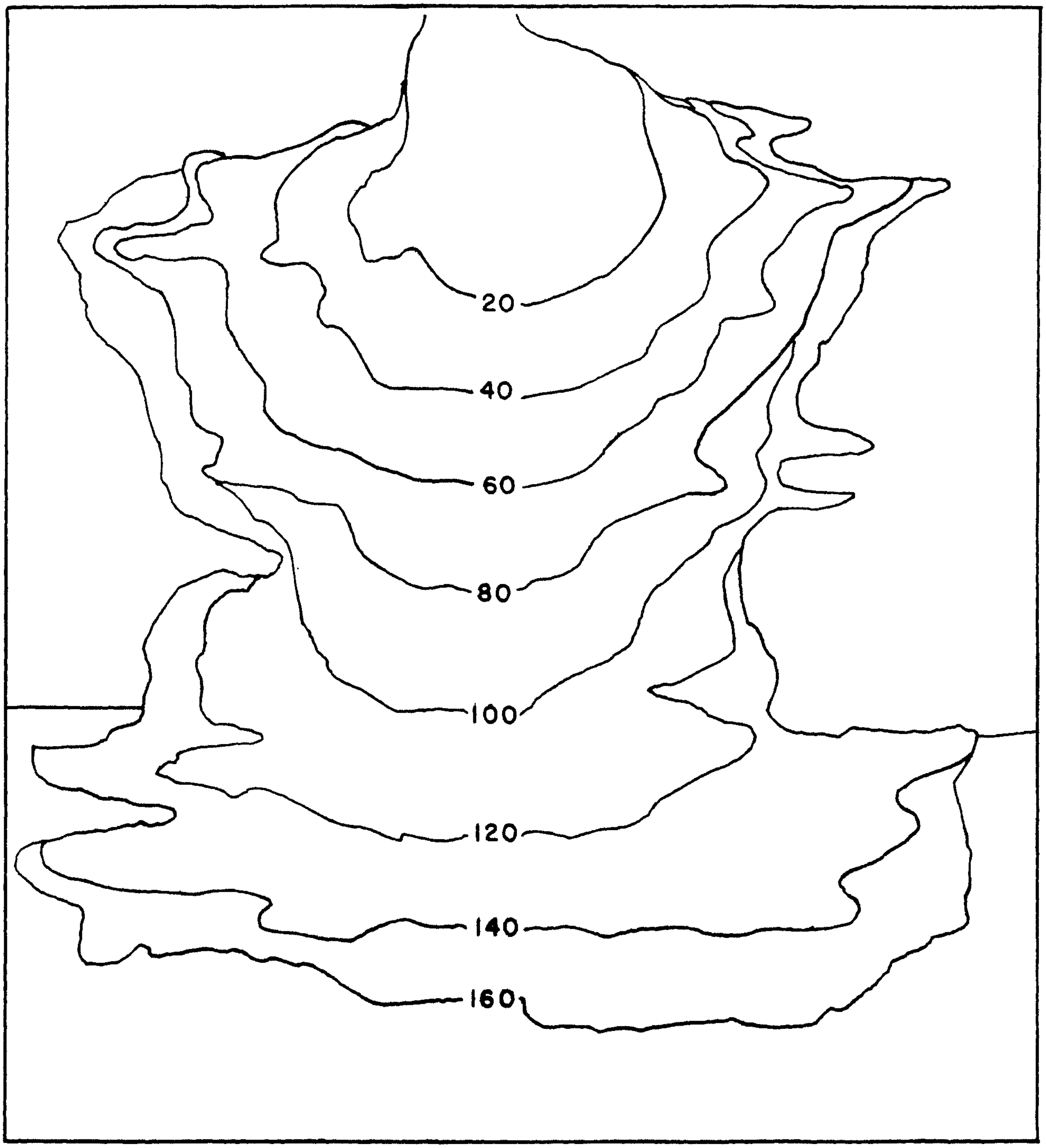

Figure 19. Movement of wetting front in damp material during test 5. Upper material is outwash, lower sediment is Ottawa sand. Numbers are minutes.

In the wet test (fig. 17) of the fourth set, the same phenomenon (as in the dry test) was observed.

From the fourth and fifth tests, it can be seen that the change in grain size from one bed to the underlying one is of considerable significance. A change in shape of the wetting front occurs at an interface between material of different permeability. Given a field situation where fine material overlays coarse material, the finer material will limit the vertical flow. Consequently, more pits would be required in this situation than those areas where finer material is overlain by coarser. With coarser material on top, one pit could have an infiltration rate that could only be equalled by possibly several pits if finer material was on top. Here again, the importance of mapping and permeability tests must be stressed.

<u>Test Six</u>: Layered material of various sizes comprised the filter material for the sixth set of tests. Values of permeability and porosity are shown in Table 3. The previously unsorted material was sieved into four sizes: 8 mesh, 10 mesh, 18 mesh, and 35 mesh, which correspond to beds 2, 4, 6, and 8, respectively (fig. 20). The tank held in ascending order, Ottawa sand (bed 1) on the bottom, bed 2, Ottawa sand (bed 3), bed 4, etc. with Ottawa sand (bed 9) on top. Bed 2 was four inches thick, bed 4 three inches thick, bed 6 two inches, and bed 8 one inch. Due to the difficulty of viewing the moving front, dye was mixed with the infiltrating water. Since the dye-water solution was mixed prior to each test, the solution may have contained less dissolved gas than was present in previous tests.

TABLE 3.

Permeability and Porosity of Selected Layers in Test Six

Bed	Permeability	Porosity
8	2285 gpd/ft^2	39.5%
6	2101 "	33.1%
4	1195 "	41.3%
2	375 "	42.5%

As water infiltrated during the dry test (fig. 20), it moved rapidly downward through bed 9 with only slight lateral spread. Upon reaching bed 8, however, downward movement was temporarily halted and the water spread laterally along the interface, partly saturating the lower part of bed 8. The wetting front crossed the interface between beds 9-8 when the head became sufficiently large and once the water entered bed 8, lateral movement in that bed became quite rapid. The water crossed the bed 7 interface after the head had increased sufficiently. After passing through the bed 7, less head was required to cross the interface between beds 7 and 6 than was necessary for the bed 8 interface. The reduction in head necessary can be attributed to the close similarity of size between the Ottawa sand and bed 6. The head required to cross the bed 5-4 interface was greater than that required to cross any of the upper boundaries. Water spread laterally only to a small degree in bed 4, and when water passed through bed 4, it did so rapidly. The water next began to saturate bed 3. Total saturation of this and the Ottawa sand in the above layer was nearly complete when the water broke through the bed 3-2 interface. The water penetrated bed 2 more rapidly than the others. No lateral movement occurred in bed 2.

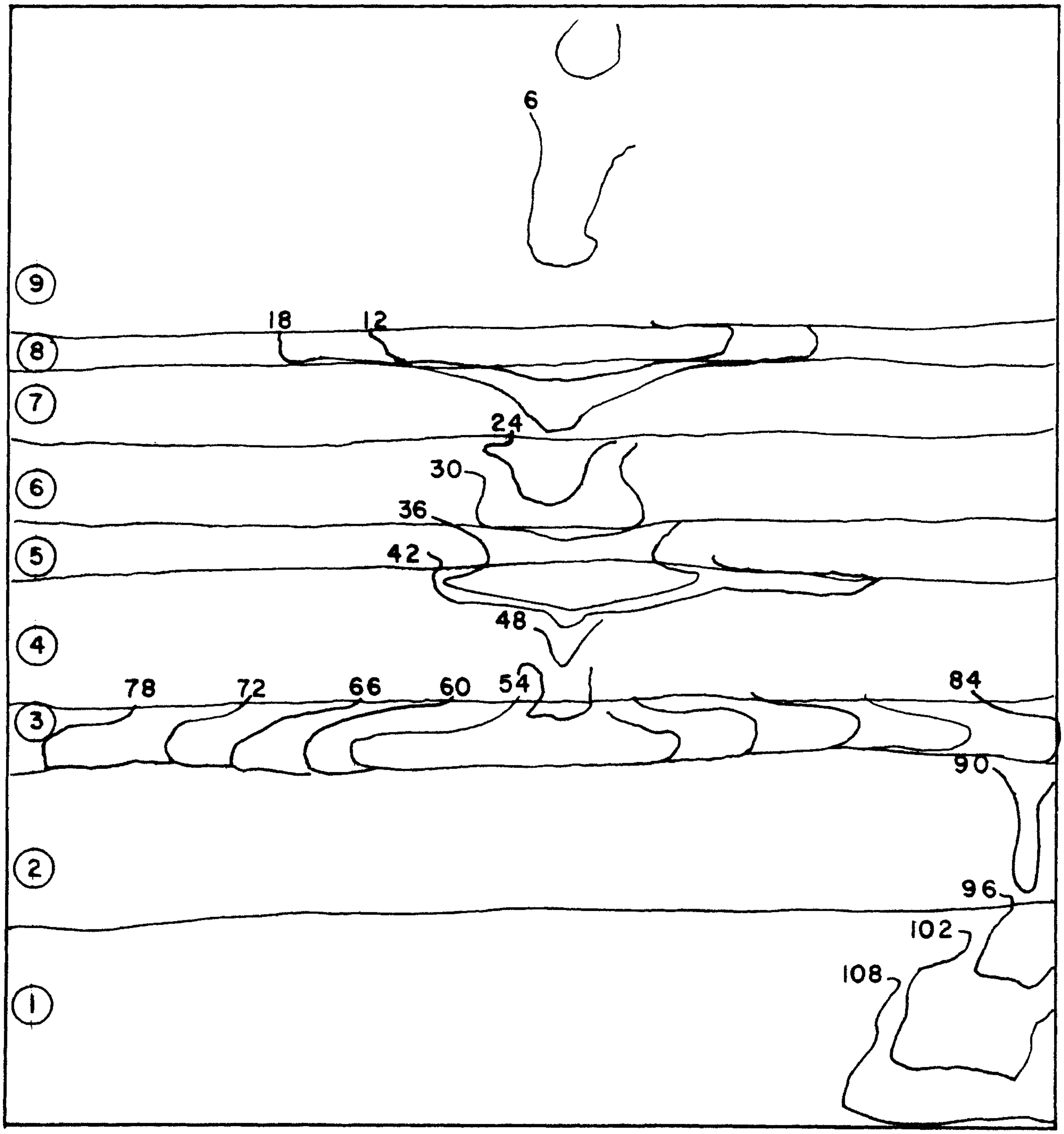

Figure 20. Movement of wetting front through dry stratified material during test 6. Numbers are minutes.

After six hours of operation, total saturation had not occurred in the bottom bed (bed 1) or in beds 8, 6, 4, or 2. Only the remaining Ottawa sand in beds 9, 7, 5, and 3 had become nearly saturated due to capillary action. Bed 8 had more of its volume saturated than bed 6, 6 more than 4, and 4 more than 2.

In the sixth set of tests, the importance of knowing the permeability of the material and its vertical and horizontal extent is demonstrated. The movement of water through the stratified material contained in the model provides a visual means of observing and measuring all the characteristics of flow beneath a recharge pit, interacting with each other.

The dry tests simulated conditions found above the water table. Of course, extremely dry conditions, as the dry tests were, are not ordinarily the case in areas having a humid climate or in areas with a water table near land surface. The wet tests simulated conditions near the water table in the capillary fringe. It is analogous to a zone of infiltration where the water table has been recently lowered and only a minimum amount of gravity drainage of the deposits has occurred.

If a field situation could be duplicated in the laboratory model, quantitative data could be calculated which would allow precise planning and operation of the recharge pit based on the qualitative flow characteristics determined in this report.

The material presented in this section was taken largely from an unpublished research report, "Subsurface Flow Characteristics of Recharge Pits as Determined from a Laboratory Model," that was prepared by T. R. Schultz. During the investigation, Mr. Schultz was an undergraduate student in the Department of Geology.

MODEL STUDY ON THE SHAPE OF A CONE OF RECHARGE UNDER VARIOUS GEOLOGIC CONDITIONS

During another model study phase of this investigation, two major types of recharge wells were installed in a large ground-water model. The clastic material filling the model was changed several times in order to represent various geologic conditions. Water was injected into each recharge well and the configuration of the water table was determined at various intervals of time. The position of the water table was monitored by an electrical probe or a pressure transducer. The object was to determine the shape of the cone of recharge around a single well.

The large laboratory model used during the course of this study had been constructed previously. It was used only briefly during an investigation of ground-water contamination in Morrow County, Ohio, in a project funded by the Office of Water Resources Research through The Ohio State University's Water Resources Center (Project No. A-004-OHIO). Much of the design, construction and testing of the model was conducted by Mr. Ted Clark, formerly an undergraduate student in the Department of Geology. The description of the model construction details and problems, taken directly but modified from Clark's original report, follows (Lehr, 1969, pp. 55-77).

The model, excluding the outside dimensions of the steel frame, is 8 feet long, 4 feet wide and 4 feet deep.(fig. 21). Within its plexiglas shell are housed two end tanks, two water inlets, two telescope drains, and 120 cubic feet of unconsolidated sand weighing approximately 10,000 pounds. The entire model, under operating conditions, weighs approximately 16,000 to 18,000 pounds.

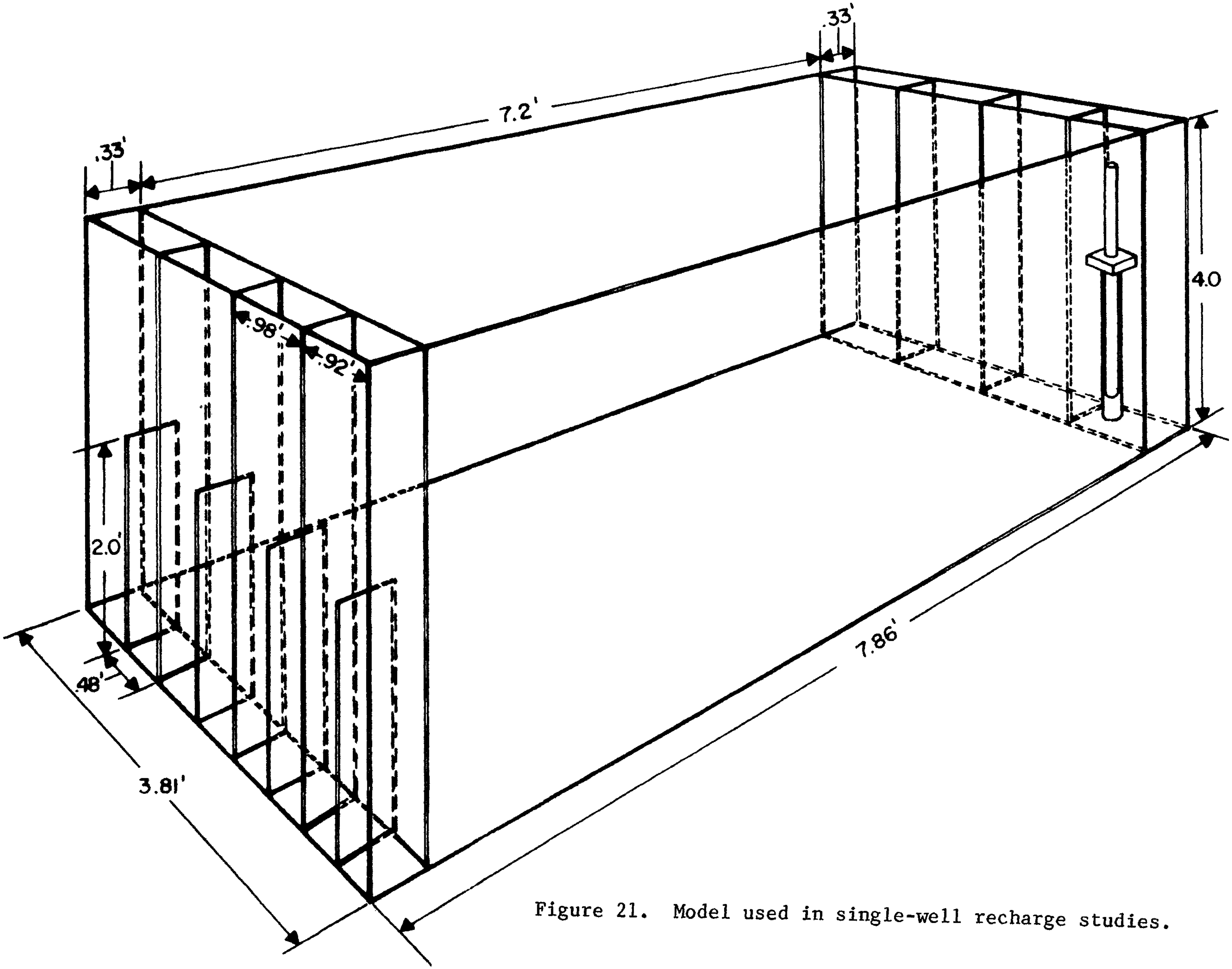

Figure 21. Model used in single-well recharge studies.

The model is designed so that water can flow through the model in either direction along its 8-foot length. This made it possible to create a gradient that can be inclined in either direction and with the potential of creating a theoretical slope ranging from 0 to 15 degrees. A 15 degree slope is a 33.3 per cent gradient. This is a drop of 1 inch in every 3.88 inches.

The steel supporting structure was designed and constructed to meet the pre-determined structural requirements. The design of the steel frame was worked out by a student in the College of Engineering at The Ohio State University. The steel frame was constructed by a commercial welding company. The steel supporting structure is lined with 1/2-inch thick sheets of plexiglas. (Plexiglas is a trade name for acrylic plastic that was used in the construction of the model.) The size of the model was limited to 4 x 8 feet, the maximum size of the plexiglas sheets.

The second phase of construction was the installation and sealing of the plexiglas sides and bottom. Due to the imperfect construction of the steel frame, the bottom and two side sheets of plexiglas were trimmed to fit. The two end sheets, which fit between the side pieces (fig. 21) were also trimmed so that the corners met with little or no space between them. The same trimming was required for the two end-tank divides.

Once the sides and bottom were trimmed and fitted in place, the problem of sealing them into position, and making the seal watertight, was undertaken. It was realized that with the model full of water and sand, considerable pressure would be exerted at all points on the plexiglas, causing some movement of the plexiglas at points of intersection. This necessitated a flexible type of sealer.

A silicone sealer was found to work the best. It proved to be quite elastic, could be removed with ease, and was not subject to any kind of biological growth. It was found necessary to abrade the surface of the plexiglas at all joints insuring a better bond. Actually, the silicone sealer was the third type used. The first, a marine sealer, proved not to be sufficiently elastic and cracked as a result of movement of the plexiglas when under pressure. The second sealer used was an aquarium sealer, very similar to roofing tar. Once set, it also cracked under expansion pressures. It was also subject to biological growth, a process not desired in this type of model. The attempts with the aquarium sealer proved an important benefit in that it indicated the points of maximum expansion. These points were subsequently shimed, thus eliminating an estimated 75 per cent of the movement due to pressure expansion. This was considered an important phase in the sealing process in that the possibility of future leaks was greatly reduced.

Design and construction of the end tanks and water circulating system constituted the third phase of construction. The end tanks are chambers at each end of the model. All water enters and leaves the model through the end tanks. From the input end tank, water passes into the model through a network of screen-covered holes distributed over the surface of the end-tank divider that separates the sand from the end tank reservoir. Two-hundred, 1/4-inch holes in each end tank divider allow for water to move in, or out of the model with uniform distribution and a minimum of turbulence.

The inside width of the end tanks is 3-3/4 inches and the tank divider was designed so that the entire assembly could be removed if necessary to do repair work in the bottom of the end tanks. This design was used because, with the exception of the silicone sealer to make the model watertight, most other joints were sealed with a solution that forms a permanent bond.

Supporting structures were cut and sealed into position for both end tanks. Five full-length supports were permanently sealed to the model, and four 1/2 length supports were permanently sealed to the end tank dividers (fig. 21). The two outside supports were permanently bonded to the sides of the model. The three center full-length supports, each containing 30 quarter-inch diameter holes to permit cross flow in the end tank, were sealed to the outside wall of the model. The four 1/2-length supports, each perforated by 15 holes and raised 1 inch above the bottom, were sealed to the inside face of the end tank divider. With this design, it is possible to remove the end tank divider. It is held in place by the pressure of the sand, which forces the divider against the supports.

The water-circulation system was also designed to allow for quick and simple replacement of parts. Two 1-3/4-inch holes were cut into the bottom sheet of plexiglas, inside each of the end tanks at the locations indicated in Figure 21.

Each hole was taped and a 1-3/4 x 1-1/4-inch bushing was sealed into each of the four holes. The bushings (one in each end tank) that are to be used as drains were shortened so that they would fit flush with the bottom surface of the model to allow for complete drainage. During operation of the model, these drains are used as water inlets. They are used as drains only when emptying the model completely, because the telescope drains cannot drain below 26 inches.

Into the other two drains is a telescope-drain assembly. This drain design allows for any overflow level desired. The inside tube can be moved up or down and is supported by two felt guides. All four of the drains can be removed for repair or replacement by removing the bushing. Since the telescope tubes are not sealed, they can be moved through the hole in the bottom of the tank.

Water enters the "in" tank through the regular drain. A constant head can be maintained by allowing the excess water to drain out through the overflow drain in the "in" tank. Water moves from the "in" reservoir tank through the model, into the "out" tank and then out through the telescope drain, which is set at a level to establish the desired gradient through the sand-filled model.

Problems encountered during construction of the model were solved at the time of construction. The first major problem encountered was that of sand flowing into the end tank between the edge of the end-tank divider and the side wall of the model. The water pressure and weight of the sand tended to spread the gaps, (which were closed before the model was filled with water and sand), thus providing a path for the sand to flow into the end tank. The situation was corrected by laying a bead of silastic along the entire contact.

The second problem encountered was that of entrance loss observed between the reservoir tank and the model. The causes of this problem were twofold. The major contributor to the entrance loss was the lack of holes in the upper end of the end-tank divider. No holes were drilled in the upper 10 inches because it was assumed that there were enough holes in the lower 4/5 of the divider. It was not the quantity of holes, however, that was causing the entrance loss but their location. With no holes at the top of the divider, water had to move into the model at a lower level and then move up through the sand to the same head as in the end tank. However, due to the flow pattern generated by a lower level discharge point in the other end of the model, the water did not move up, but flowed away from the end tank divider towards the discharge point, which is at a lower hydraulic head.

The entrance-loss problem was reduced by the addition of a network of holes drilled in the upper end of the end-tank divider. The spacing of the holes in the upper part of the divider was increased to about one per square inch as opposed to one per four inches in the lower part of the divide. This addition of holes at the top end allows more water to move directly into the uppermost layers of the model and, thus, reduce entrance loss. As with the first network of holes, the additional holes were covered with 60 mesh screen.

The problem of sand stratification and pockets in the sand filled with water or air was also encountered. Most of the sand was poured into the model, which was partially filled with water. This was done in the hopes of preventing air pockets that could result from filling the model with dry sand. Air pockets would tend to form around wells and at other critical locations. Having the model filled with water prevented most of the air from becoming entrapped, and also aided in spreading the sand. Unfortunately, the water also tended to sort the sand and form thin layers of finer material. The thin layers show up as cross-bedding and their extent are dependent upon the distribution of the sand at the time of pouring.

The problem was, at least partly, solved by filling the rest of the model with dry sand. The sand, once poured into the model, was then spread by hand to insure even and uniform distribution. The problem of the cross-bedded and sorted sand was somewhat eliminated by means of reworking the sand with a water hose inserted into the stratified sand.

<u>Testing, Experiments and Potential Uses of the Model</u>

The initial test was involved with the development of a water-table gradient in the model. This was accomplished by means of water inflow at one end of the model and a discharge at the other end. This test indicated the problems related to high permeability of the clastic material and the entrance loss at the "in" reservoir. Well pumping recharge tests, and the resulting shallow cones of depression or recharge confirmed the high permeability situation.

The high permeability of the model, however, has little effect on the recharge experiments. The recharge mound around the recharge well which develops during the tests, is not as exaggerated as it would be if the permeability was lower.

Description of Recharge Tests

Experiment 1.

The model was filled with Ottawa sand to within one foot of the top. A sand point (recharge well) was inserted in the center of the model to a depth of 25 inches. Deaired water was pumped into the recharge well at a rate of 4.25 gpm for 12 hours. The potential was measured at the end of 12 hours in 5 observation wells at several different depths using a pressure transducer. The measured shape of the cone of recharge is shown in figures 22 and 23.

The experiment is analogous to a screened recharge well that penetrates two-thirds of a homogeneous water-table aquifer. The shape of the cone of recharge surrounding the model well closely resembles a theoretical cone that is based on the Theis equation. Within three inches of the recharge well, however, there is a change in slope on a semi-log plot that may represent turbulent flow in the well vicinity.

Following this experiment, the sand point was no longer used mainly because it was difficult to control the rate of recharge and maintain steady-state conditions. In addition, as water cascaded down the well it became aerated, which, in turn, resulted in bubbles forming around the well and in

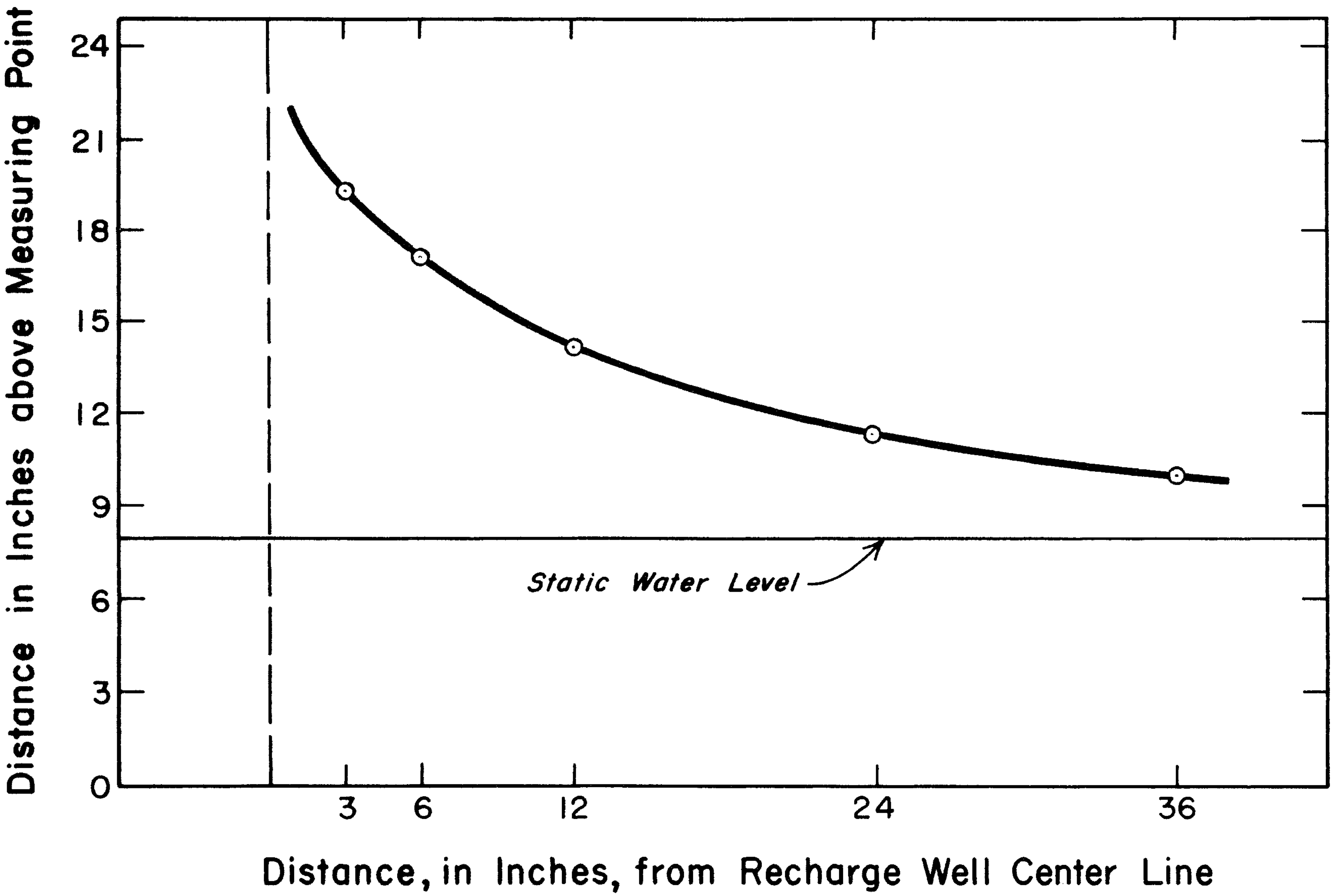

Figure 22. Shape of the cone of recharge during experiment 1.

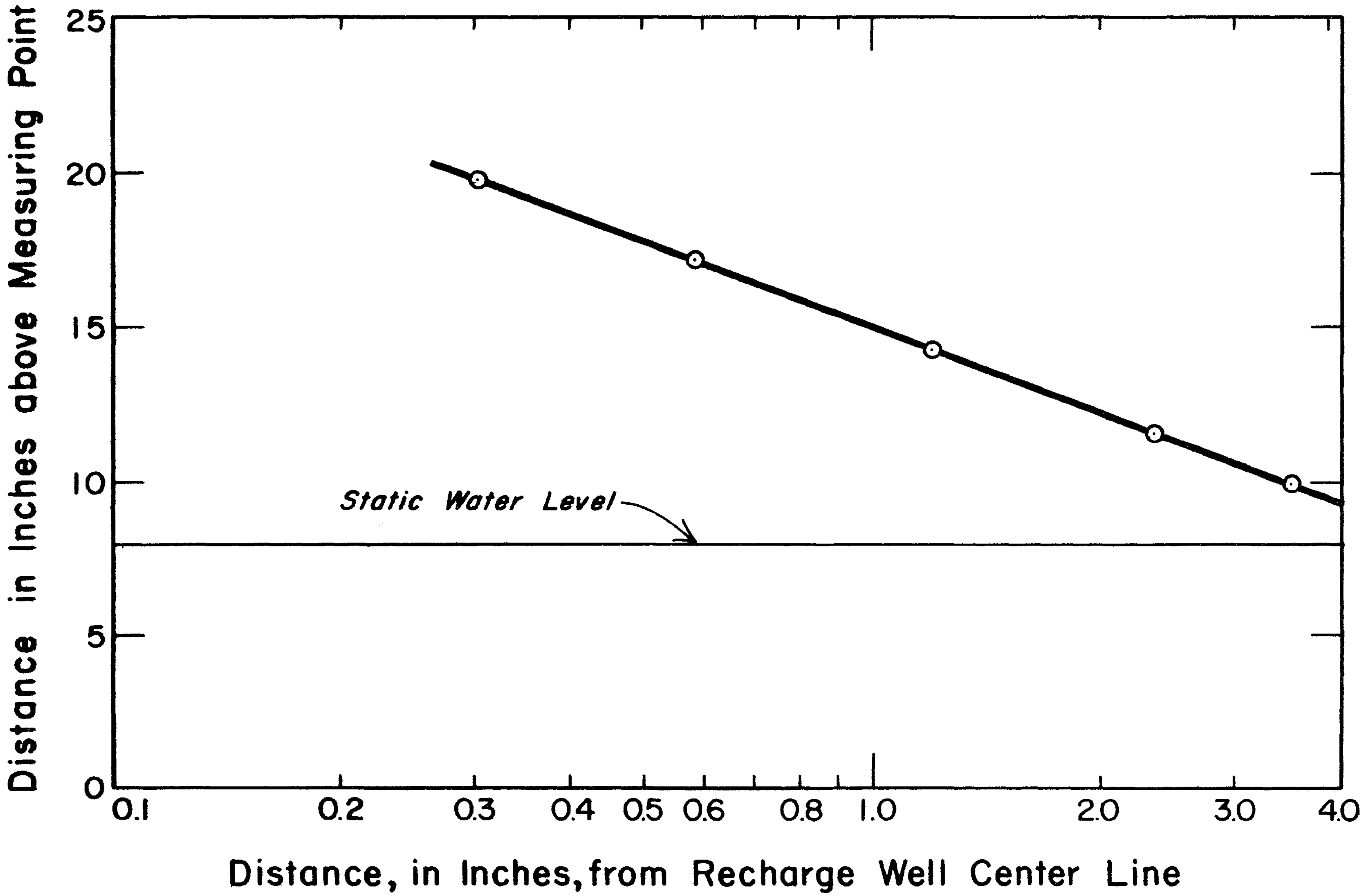

Figure 23. Shape of the cone of recharge during experiment 1.

observation wells. The extreme sensitivity of the transducer system also created several problems.

Experiment 2.

The model was filled to within a half foot of the top with homogeneous Ottawa sand. A plastic cylinder, 9 inches in diameter and 1 foot long, was inserted in the center of the model to serve as a recharge well. Deaired water was pumped into the recharge well at a constant rate (2.75 gpm), and the water level was measured and potentials determined at several depths in 9 observation wells. The major objective in this experiment was to calibrate the pressure transducer system. Data plots are shown in figure 24.

This experiment is analogous to a large diameter, shallow recharge well or pit that has been excavated in a homogeneous, unconfined aquifer. The shape of the cone of recharge surrounding the well closely approximates theoretical conditions.

Continued work with the transducer system showed that it was much too sensitive for experiments of this type and that the head-transducer readout relationship was not linear. Consequently, the method was abandoned and replaced with an electrical resistance probe designed in the laboratory specifically for the problem at hand. The probe technique is much faster than the transducer method previously used and provides data of greater accuracy and reliability on the position of the water table. A comparison of the shape of the recharge cone based on transducer and probe data are shown in Figures 25 and 26.

Experiment 3.

In order to more effectively utilize the model system, the recharge well was again changed. During the remaining series of tests this final type

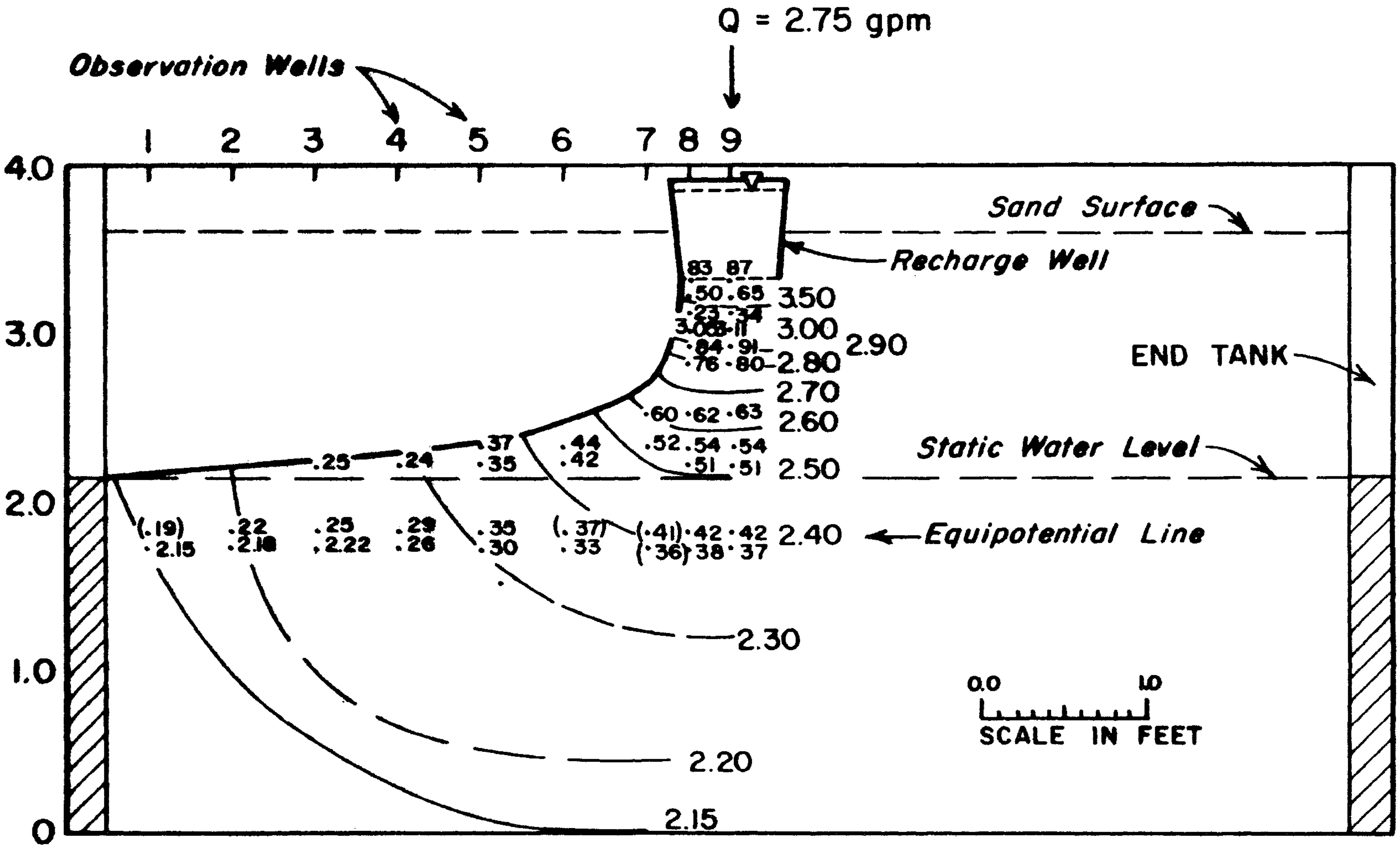

Figure 24. Shape of the cone of recharge in a homogeneous media where Q = 2.75 gpm.

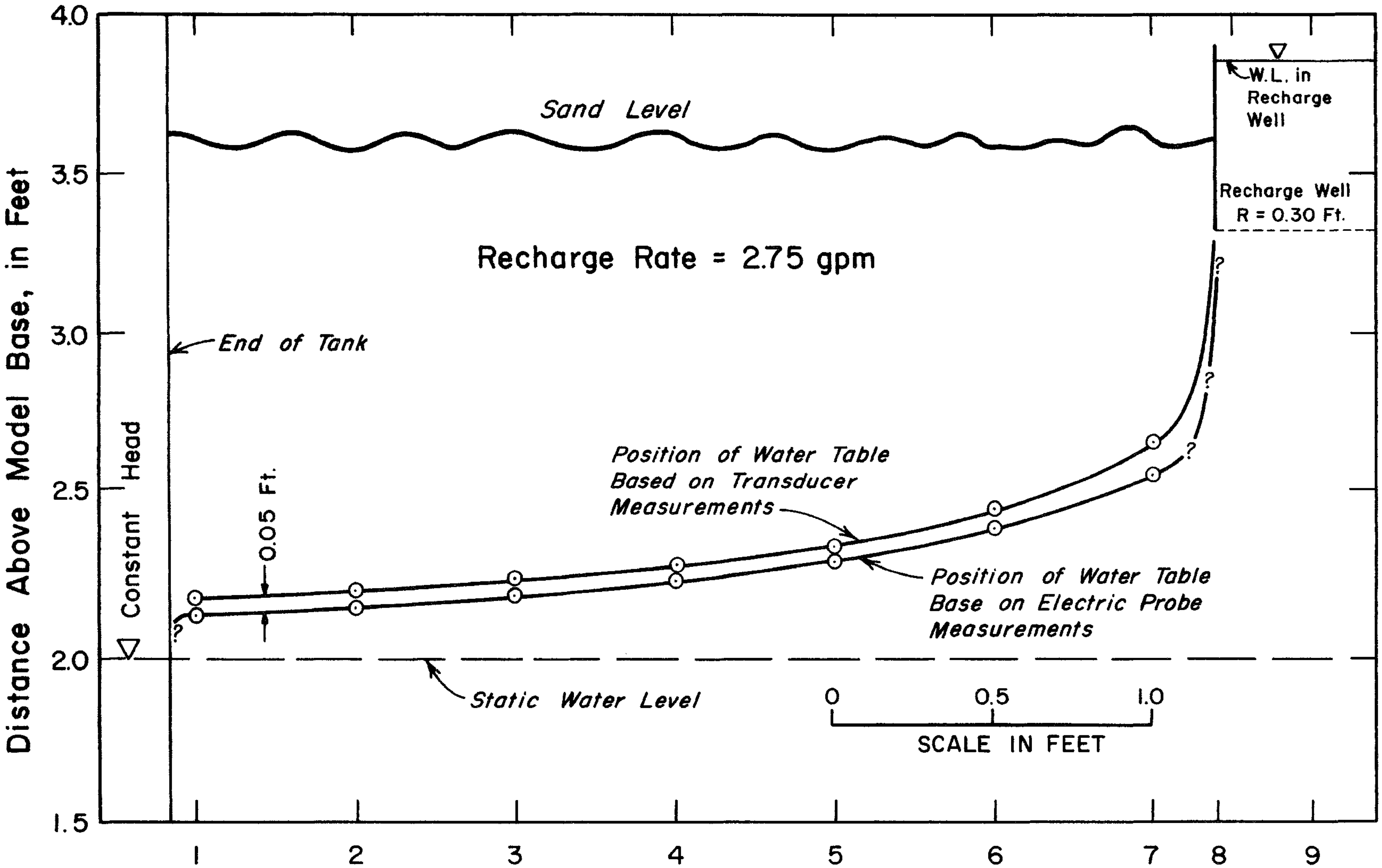

Figure 25. Shape of the cone of recharge in a homogeneous media where Q = 2.75 gpm as measured by a pressure transducer and an electric probe.

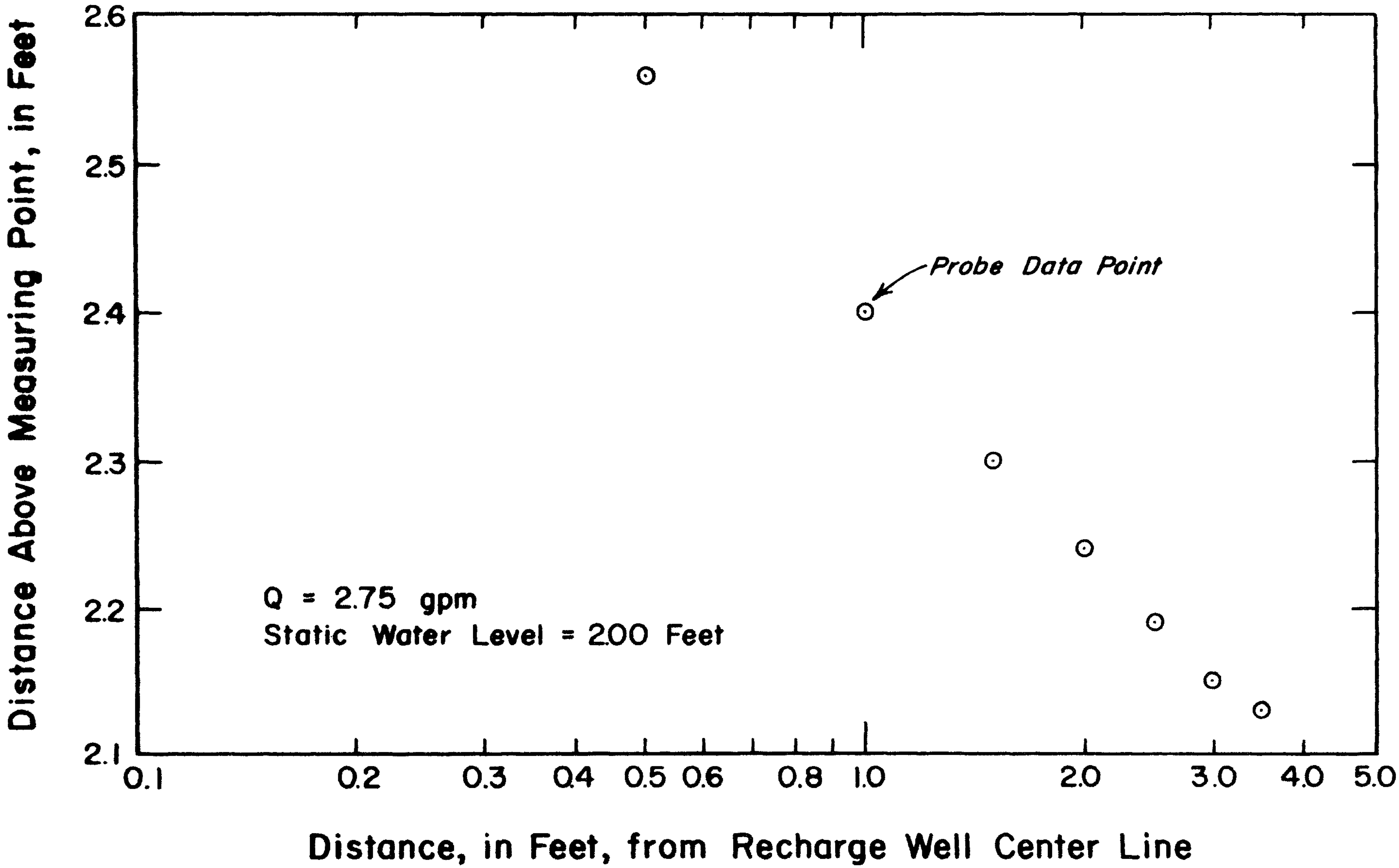

Figure 26. Shape of cone of recharge in a homogeneous media where Q = 2.75 gpm.

of well remained in use. It consisted of half of a one-foot length of plastic pipe, 0.656 feet in diameter. The half of the recharge well was attached to the side of the model in a central location. This permitted the measurement of the cone of recharge in three directions, each measuring 3.5 feet in length from the well centerline. The bottom of the well was 3.0 feet above the model base.

To restrict the flow of water from the well and reduce erosion around it, four layers fibrous tissue paper was attached to the bottom of the well in contact with a screen. In an attempt to distribute the flow more uniformly over the bottom of the recharge well and to dissipate the kinetic energy of the water cascading into it, the recharge well was partly filled with coarse ground glass. The lower 3 inches were filled with fine glass that generally ranged between 1 - 2 mm in diameter. This layer was then covered with 3 inches of medium-sized glass fragments ranging between 3 and 5 mm in diameter.

During the following series of experiments, deaired water was pumped into the recharge well at different rates and the position of the water table was measured with an electric probe. This technique permitted the determination of the position of the water table directly under the recharge well, a feat that previously had been impossible.

Six recharge tests were conducted in order to test the new system. During the tests the recharge rate was varied and for each particular test the water temperature was slightly different (table 4). The media consisted of homogeneous unstratified Ottawa sand forming a water-table aquifer.

As is evident from Figures 27 - 30, the major portion of the recharge mound exists within two feet of the recharge well centerline, that is, the most significant change in slope is nearly equal to the original saturated

Table 4. Data for Experiment 3, Tests A - F

Test A Q = 0.561 gpm T = 22.3°C

Distance, in feet, from recharge well centerline	Depth, in feet, to water surface above model base
0.5	2.24
1.0	2.16
1.5	2.11
2.0	2.10
2.5	2.09
3.0	2.08
3.5	2.07

Test B Q = 0.544 gpm T = 22.6°C

0.5	2.24
1.0	2.15
1.5	2.11
2.0	2.10
2.5	2.08
3.0	2.08
3.5	2.07

Test C Q = 1.55 gpm T = 22.0°C

0.5	2.62
1.0	2.39
1.5	2.26
2.0	2.19
2.5	2.14
3.0	2.12
3.5	2.10
2.592	2.056

Test D Q = 1.38 gpm T = 23.6°C

0.50	2.58
0.75	2.50
1.0	2.40
1.5	2.30
2.0	2.24
2.5	2.21
3.0	2.19
3.5	2.17

Test E Q = 1.65 gpm T = 23.6°C

0.50	2.65
0.75	2.46
1.0	2.45
1.5	2.33
2.0	2.24
2.5	2.22
3.0	2.19
3.5	2.18

Test F Q = 1.03 gpm T = 23.0°C

0.5	2.47
0.75	2.40
1.0	2.32
1.5	2.24
2.0	2.21
2.5	2.19
3.0	2.17
3.5	2.16

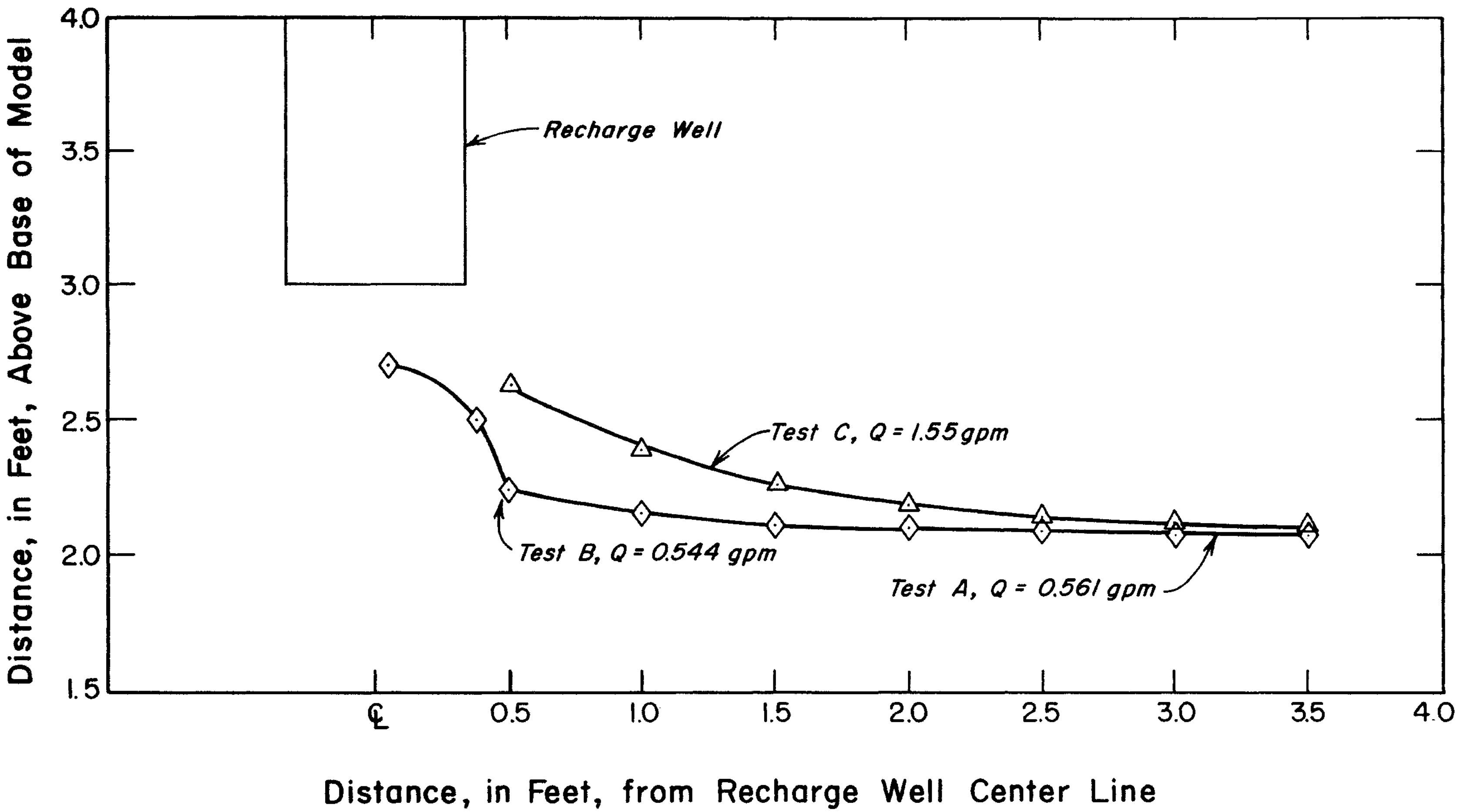

Figure 27. Shape of cone of recharge in homogeneous media where Q = 0.544, 0.561, and 1.55 gpm.

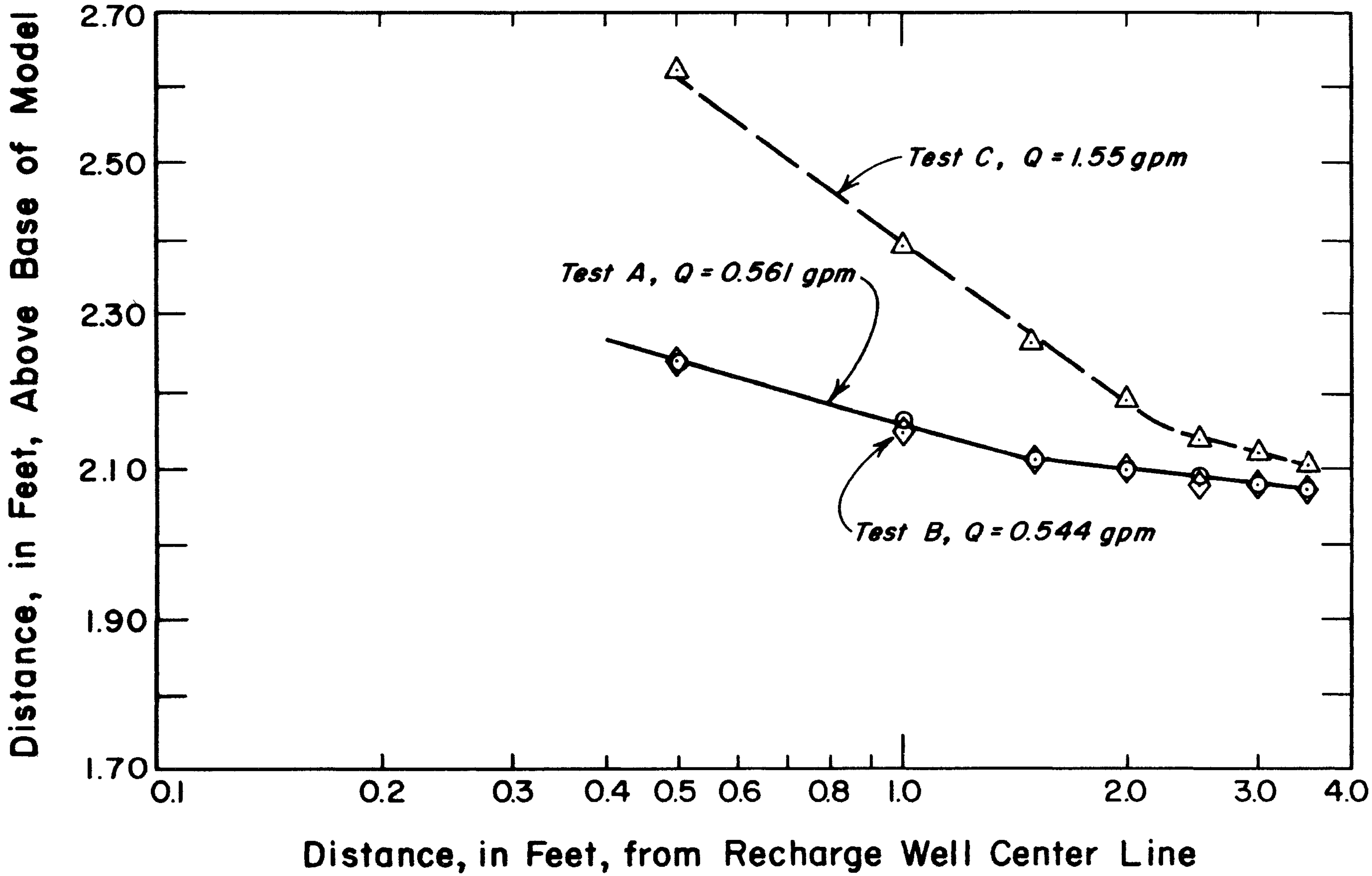

Figure 28. Shape of cone of recharge in homogeneous media where Q = 0.544, 0.561, and 1.55 gpm.

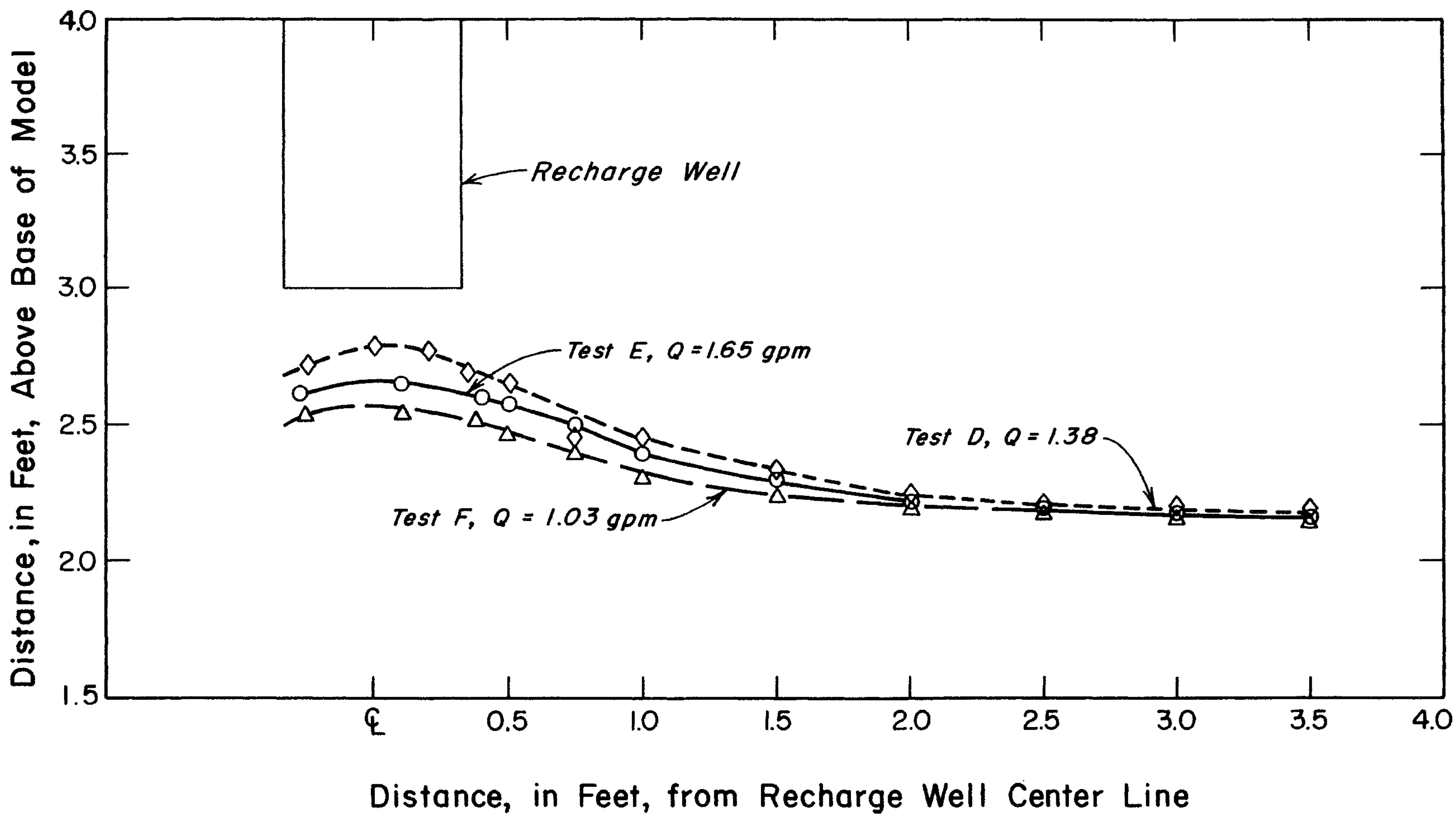

Figure 29. Shape of cone of recharge in homogeneous media where Q = 1.03, 1.38, and 1.65 gpm.

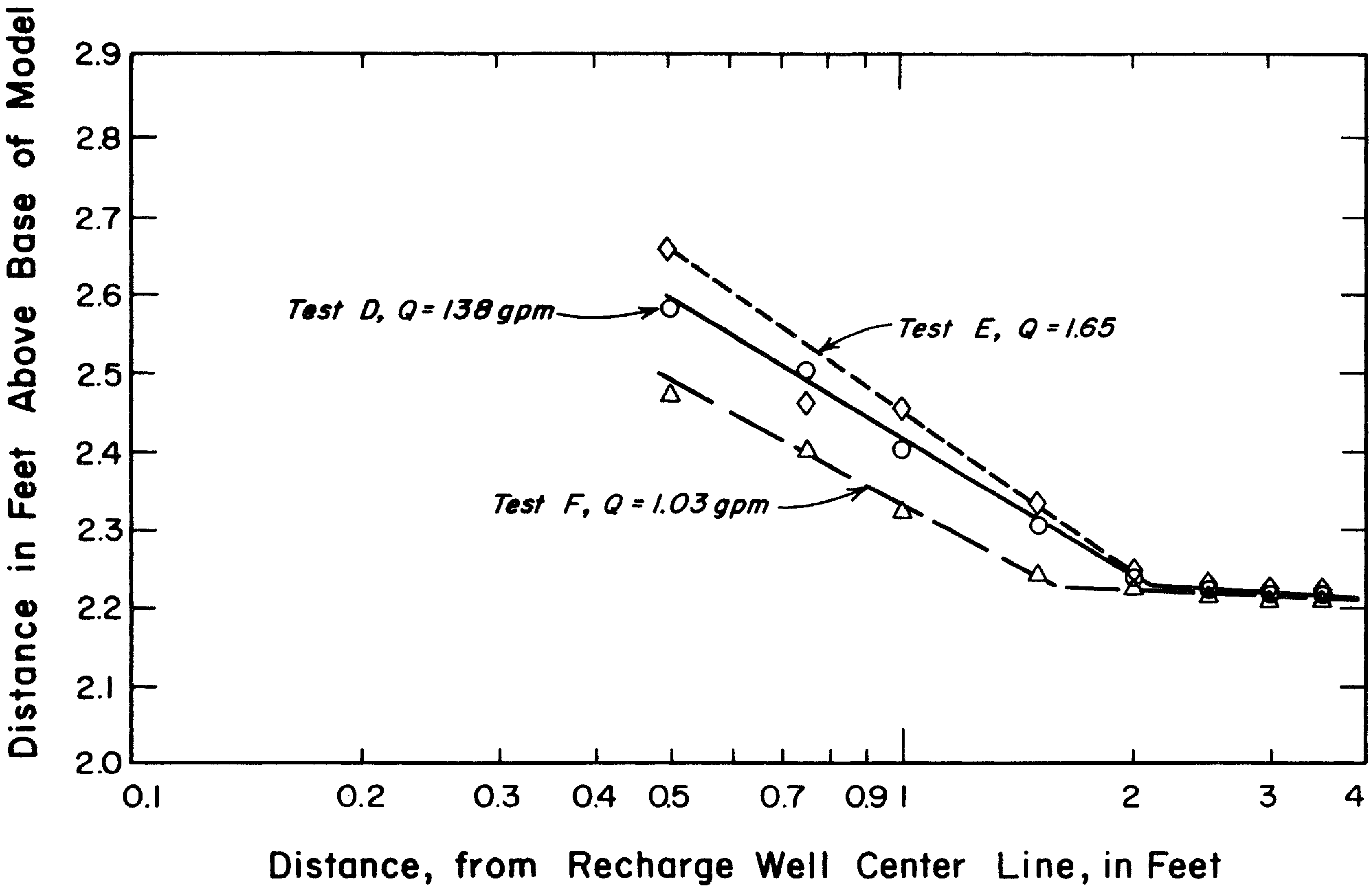

Figure 30. Shape of cone of recharge in homogeneous media where Q = 1.03, 1.38, and 1.65 gpm.

thickness of the model. Furthermore, the gradient increases substantially directly under the point source of recharge. The gradient changes are particularly evident on the semi-log plots (Figs. 28, 30). It is also evident in tests D, E, and F, that the position of the water table is nearly the same at distances exceeding two feet even though the recharge rates differ.

Experiment 4.

In order to conduct this series of experiments, which deal with layered material, Ottawa sand was removed from the model to a depth of approximately 1.66 feet below the top of the tank. The model was then refilled to about .8 foot from the top with "coarse sand." This oxidized material was obtained from a local gravel pit and ranged in size from silt to very coarse sand. It also contained a small amount of clay. The material, being unwashed, is rather typical of much of the finer-grained outwash that occurs in Ohio.

When the model was refilled, the half cylinder recharge well was again installed along the model wall with the centerline having a minimum distance of 3.5 feet from the other two walls and 4 feet from the third wall. The bottom of the well was 1 foot below the top of the tank. The well was then partly filled with crushed glass as in previous tests. Deaired water was then pumped into the recharge well and the changes in water level measured. The water level in the end tanks was maintained at a constant level 1.87 feet below the top of the model.

The objective of the remaining tests was to determine the shape of the cone of recharge in a stratified aquifer. This type of situation more nearly conforms to actual field conditions. During test A, water was pumped into the recharge well at a constant rate of 1.05 gpm at a temperature of 21.8° C. The water-level data and graphic plots are shown in Table 5 and Figures 31 and 32. During test B, the recharge rate was 0.264 gpm at a water

Table 5. Data for Experiment 4, Tests A - D

<u>Test A</u> Stratified Model (two layers)

Q = 1.05 gpm T = 21.8°C

Distance from recharge well, center line, in feet	Water level, in feet, below top of model
0.5	1.46
0.75	1.55
1.00	1.62
1.25	1.67
1.50	1.69
2.00	1.73
2.50	1.78
3.00	1.80
3.48	1.82
End tank	1.87

<u>Test B</u> Stratified Model (two layers)

Q = 0.264 gpm T = 22.1°C

0.50	1.75
0.75	1.79
1.00	1.81
2.00	1.85
3.00	1.86
End tank	1.87

<u>Test C</u> Stratified Model (two layers)

Q = 1.06 gpm T = 23.0°C

0.00	1.30
0.24	1.52
0.42	1.51
0.50	1.49
0.75	1.56
1.00	1.63
1.25	1.68
1.50	1.72
1.75	1.755
2.00	1.76
2.25	1.775
2.50	1.79
2.75	1.81
3.00	1.82
3.25	1.825
3.48	1.84
End tank	1.87

Test D   Stratified Model (two layers)

Q = 1.06 gpm   T = 21.6°C

0.18	1.49
0.38	1.53
1.00	1.615
1.25	1.66
1.50	1.71
2.00	1.76
2.50	1.78
3.00	1.81
3.48	1.83
End tank	1.87

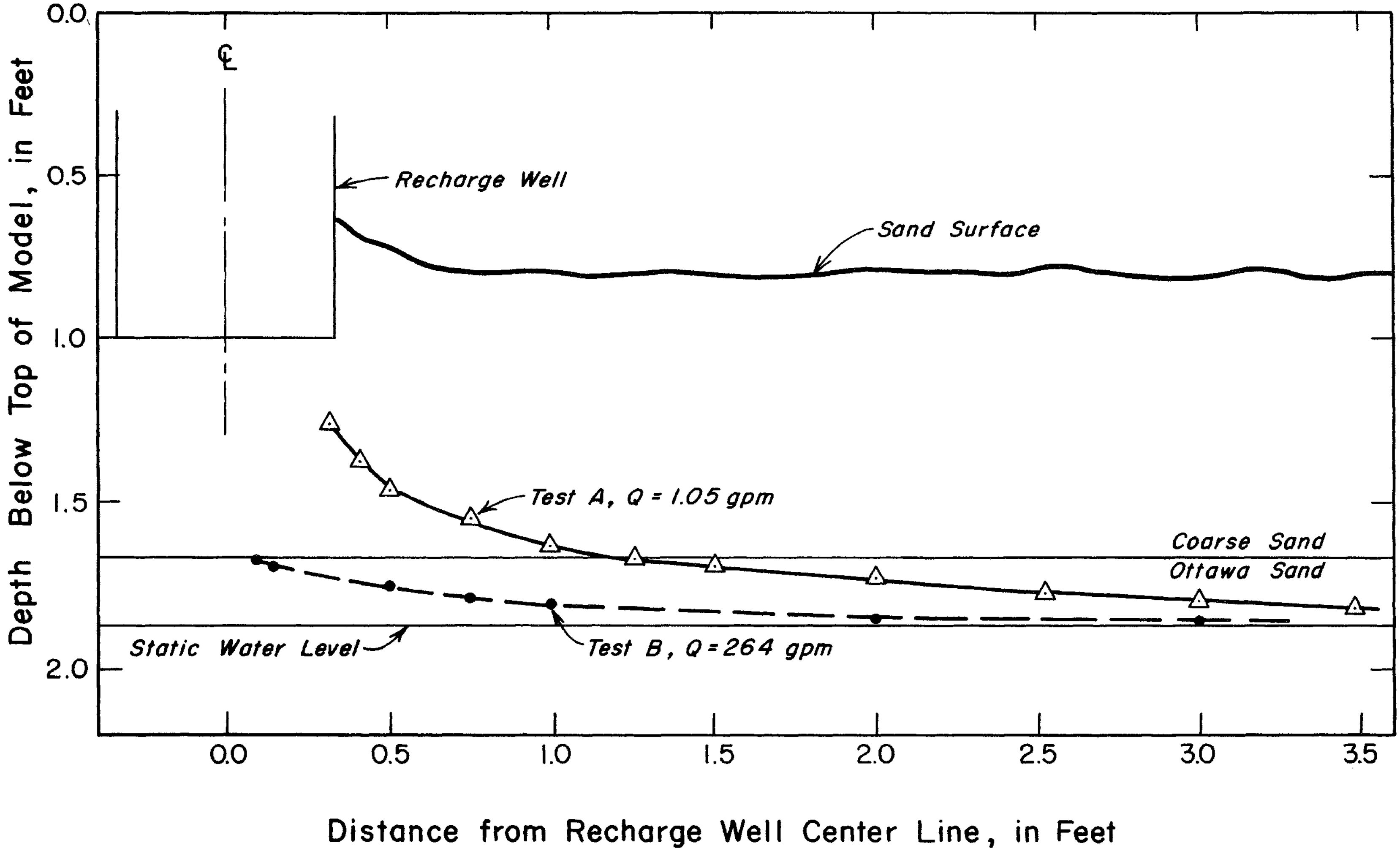

Figure 31. Shape of cone of recharge in a two-layered media where Q = 0.264 and 1.05 gpm.

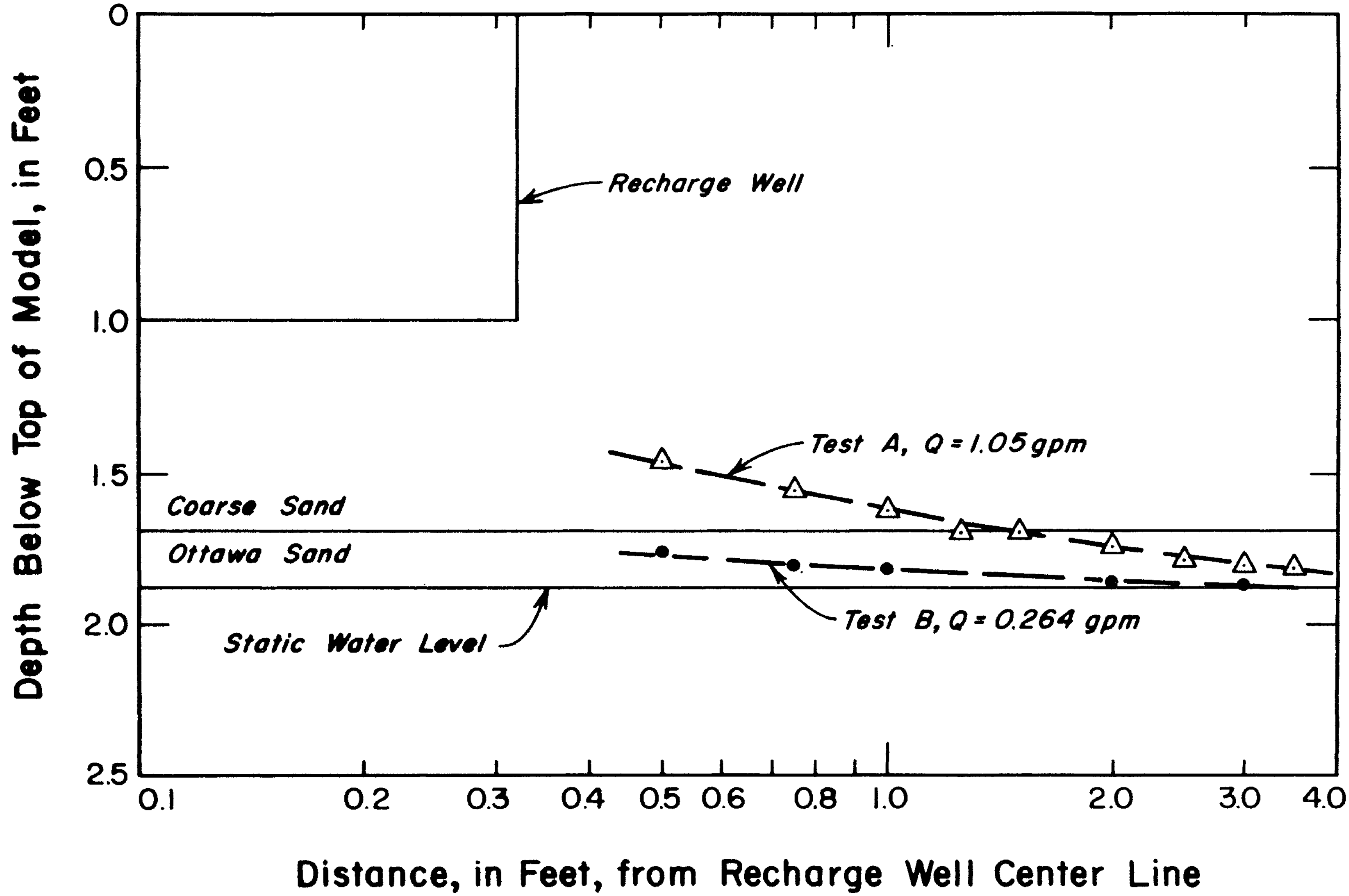

Figure 32. Shape of cone of recharge in a two-layered media where Q = 0.264 and 1.05 gpm.

temperature of 22.1°C. The resulting data also are shown in Table 5 and Figures 31 and 32.

The recharge rate during test A was sufficient to cause the water level to rise above the coarse sand-Ottawa sand contact. Although not readily apparent on an arithmetic plot, a change in gradient in the vicinity of the coarse sand-Ottawa sand boundary is evident on a semi-log plot (Fig. 32). During test B, when the recharge rate was only 0.264 gpm, the water level was measured only in the lower homogeneous media.

During tests C and D, the cone of recharge crossed the coarse sand-Ottawa sand boundary, which caused a slight change in gradient. The major objective of these two tests, however, was to examine the cone in the near vicinity of the well and relate it to the height of the well above static water level. As is readily evident on Figure 33, the less the length of unsaturated flow, the more exaggerated the mound is under the recharge well (compare test C and D). This, in part, may explain why recharge rates in actual field conditions tend to decrease as the water table approaches the base of a recharge pit or well.

Experiment 5.

The five tests conducted during experiment 5 are similar to those in experiment 4, except that the cone of recharge is measured in two perpendicular directions. The model consists of a two-layer sequence with coarse sand overlying Ottawa sand (Fig. 35). The base of the recharge well remained throughout the experiment at a depth of .5 foot beneath the top of the model. The static water level ranged from a depth of 1.35 to 1.88 feet, depending on the objective of each particular test (Fig. 35). The rate of recharge was also changed from one test to the next.

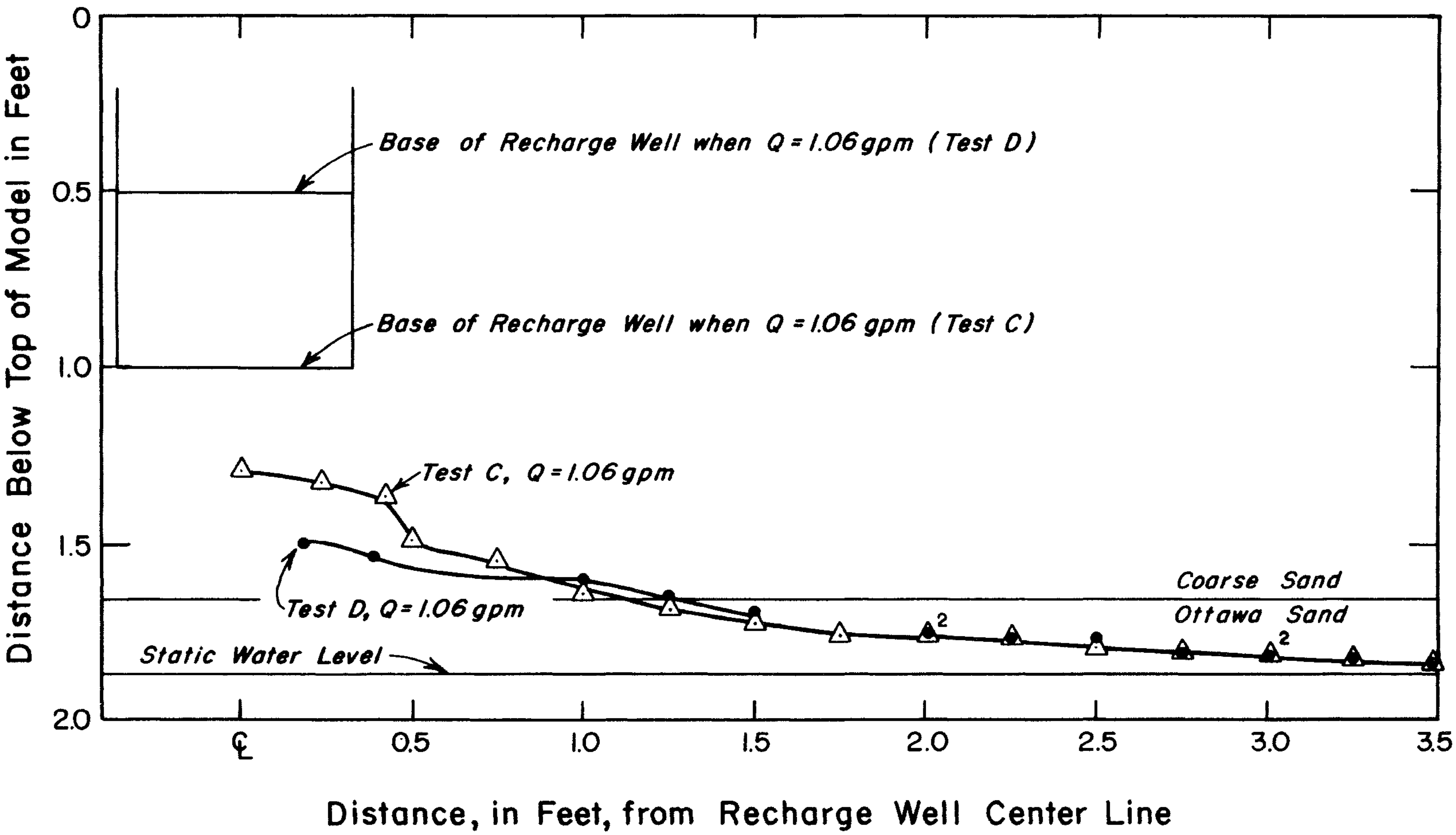

Figure 33. Shape of the cone of recharge in two-layered media where Q = 1.06 gpm and base of recharge well is 0.5 and 1.0 feet below top of model.

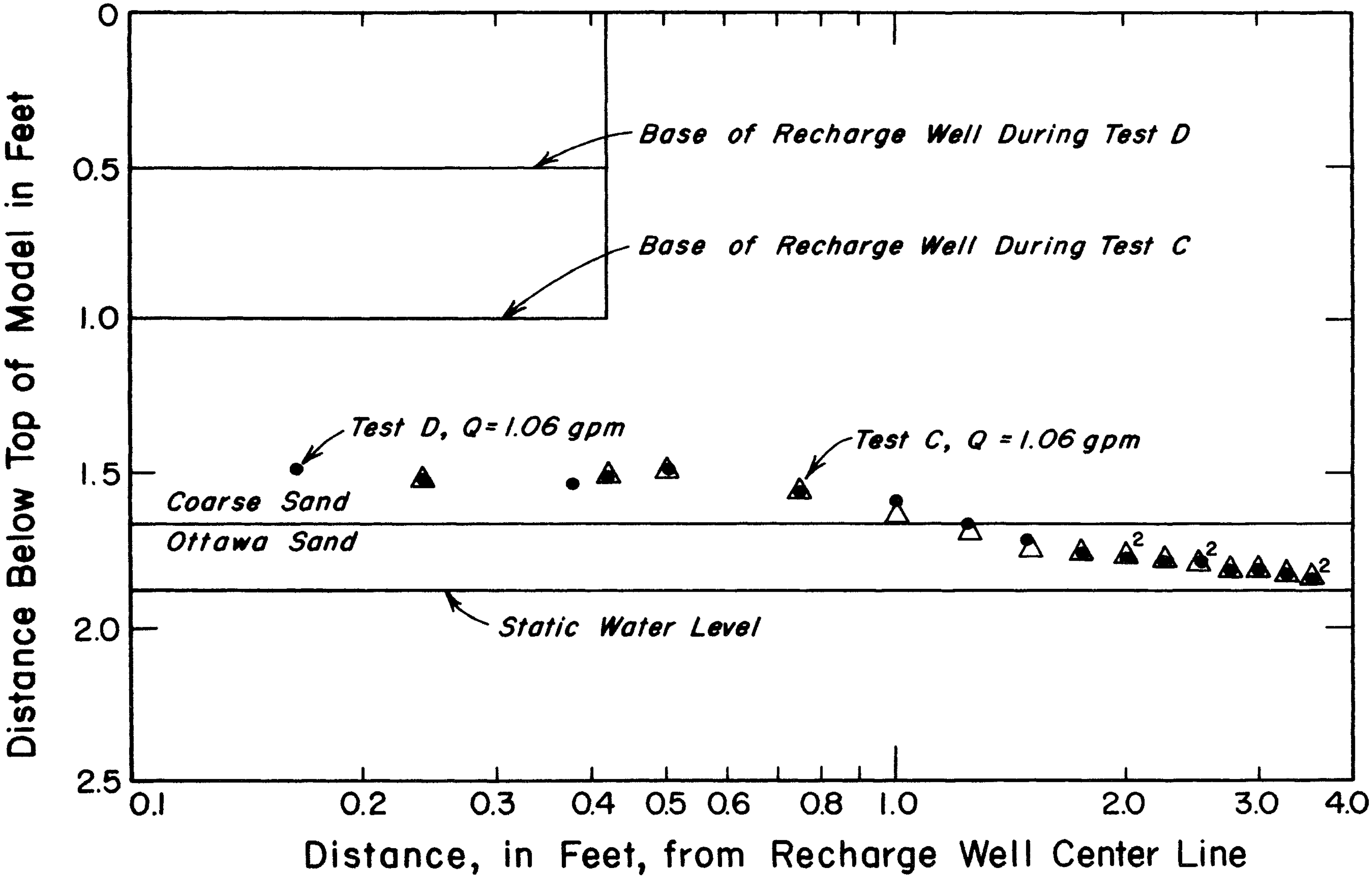

Figure 34. Shape of cone of recharge in two-layered media where Q = 1.06 gpm and base of recharge well is .5 and 1.0 but below top of model.

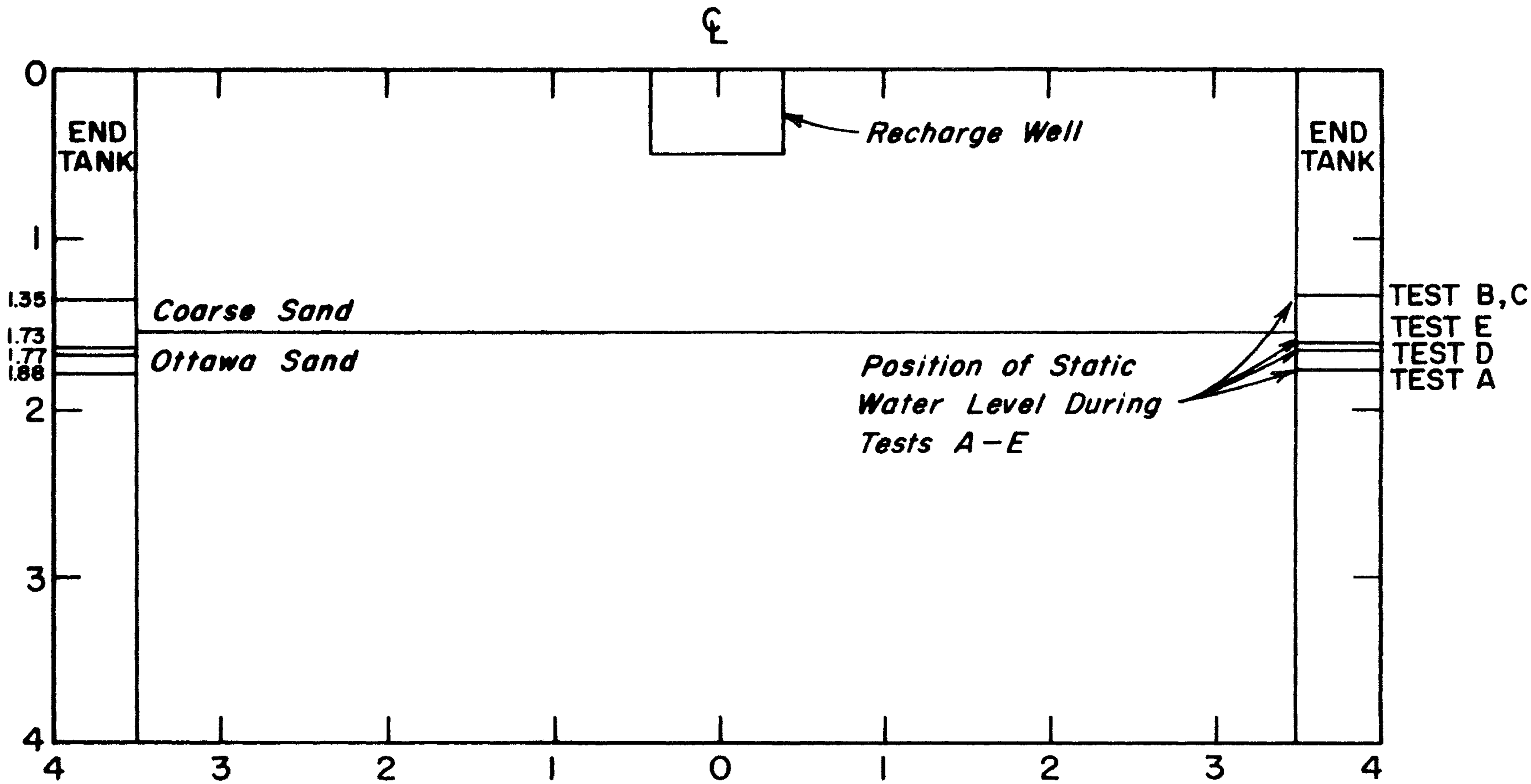

Figure 35. Model setup for experiment 5, tests 1-5.

The objective of this series of tests was to examine the cone of recharge under several sets of conditions:

Test A. Static water level maintained in the lower unit (Ottawa sand) and measured in perpendicular directions at a low recharge rate.

Test B. Static water level maintained in the upper unit (coarse sand) and measured in perpendicular directions at a low recharge rate.

Test C. Static water level maintained in the upper unit (coarse sand) and measured in perpendicular directions at a high recharge rate.

Test D. Static water level in lower unit but recharged at such a rate that the cone extends across the boundary and measured in perpendicular directions at a low recharge rate.

Test E. Static water level in lower unit but recharged at such a rate that the cone extends across the boundary and measured in perpendicular directions at a high recharge rate.

The cone of recharge was measured in two directions (west from centerline and north from centerline) in order to evaluate model boundaries. For example, end tanks exist on only the east and west sides of the model and act as discharge points. Consequently, the water level should be lower in the east or west direction in comparison to the north direction (the recharge well is attached to the south side of the model).

Test A. During this test the static water level was 1.88 feet below the top of the model and in the lower unit. Water, at a temperature of 24°C. was pumped into the recharge well at a constant rate of 0.29 gpm. The cone reached equilibrium within three hours (Table 6). Data points in both north and west directions generally closely conform, but as expected, extreme points in the north direction are slightly higher than points at a similar distance in the west direction (Figs. 36 and 37). The extent of the cone of recharge was slightly in excess of the original saturated thickness of the model.

Test B. The static water level in test B was raised at 1.35 feet below the top of the model at a position in the coarse sandy material. Water was pumped into the recharge well at a constant rate of 0.26 gpm. The cone reached equilibrium within four hours when the water level was measured (Figs. 38 and 39). Nearly all the data points in both directions are the same (Table 6). The exception occurs, however, as expected near the model margins with the level reaching static within 3 feet of the recharge well centerline. The radius of the cone of the recharge in the model is just slightly greater than the original saturated thickness.

Test C. During rest C, the static water level was maintained at the position established in test B (1.35 feet). Water was pumped into the recharge well at a constant rate of 1.07 gpm. When steady-state conditions were achieved, the position of the water table, which was limited to the upper sequence, was

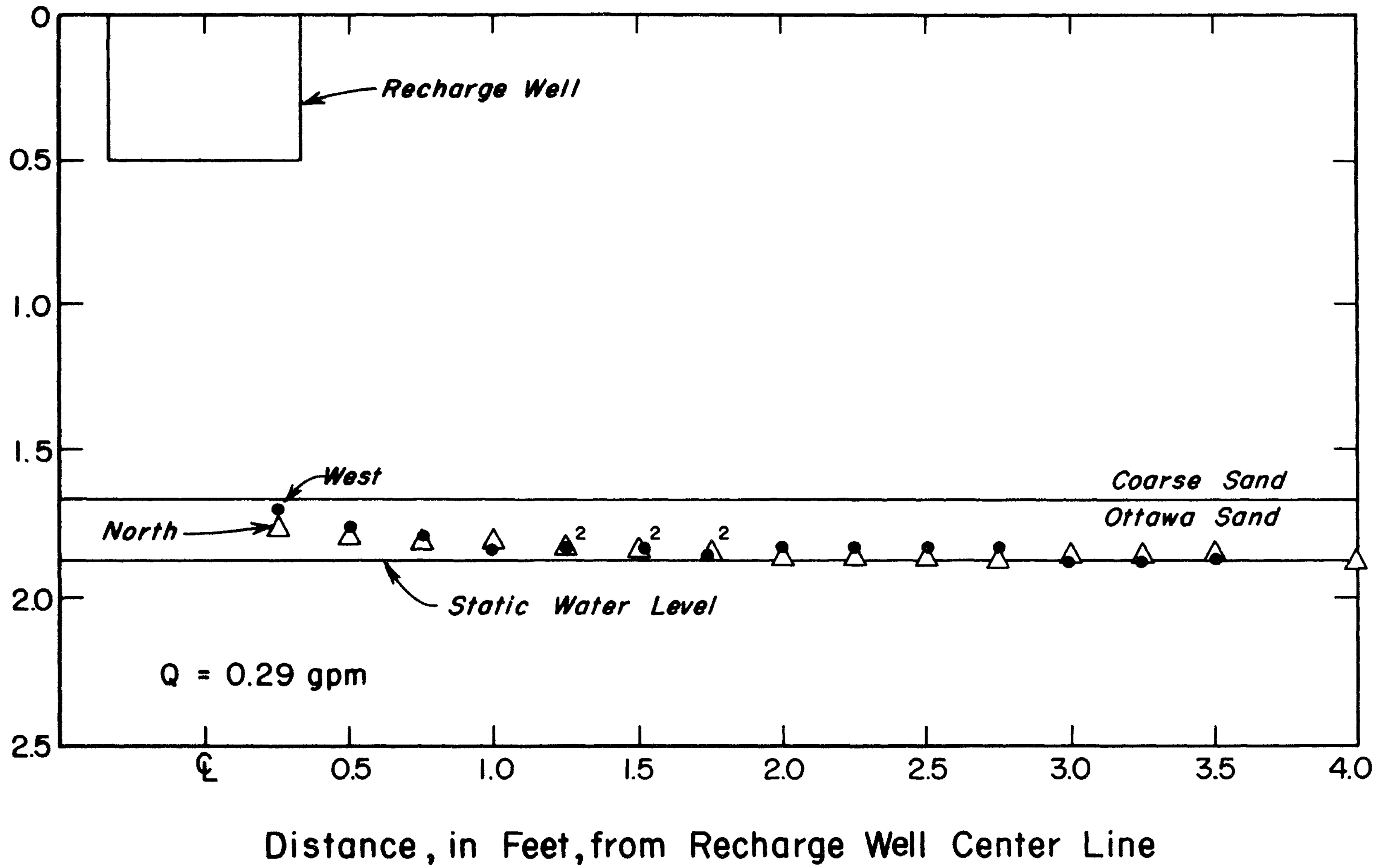

Figure 36. Shape of cone of recharge where Q = 0.29 gpm in a two-layered sequence.

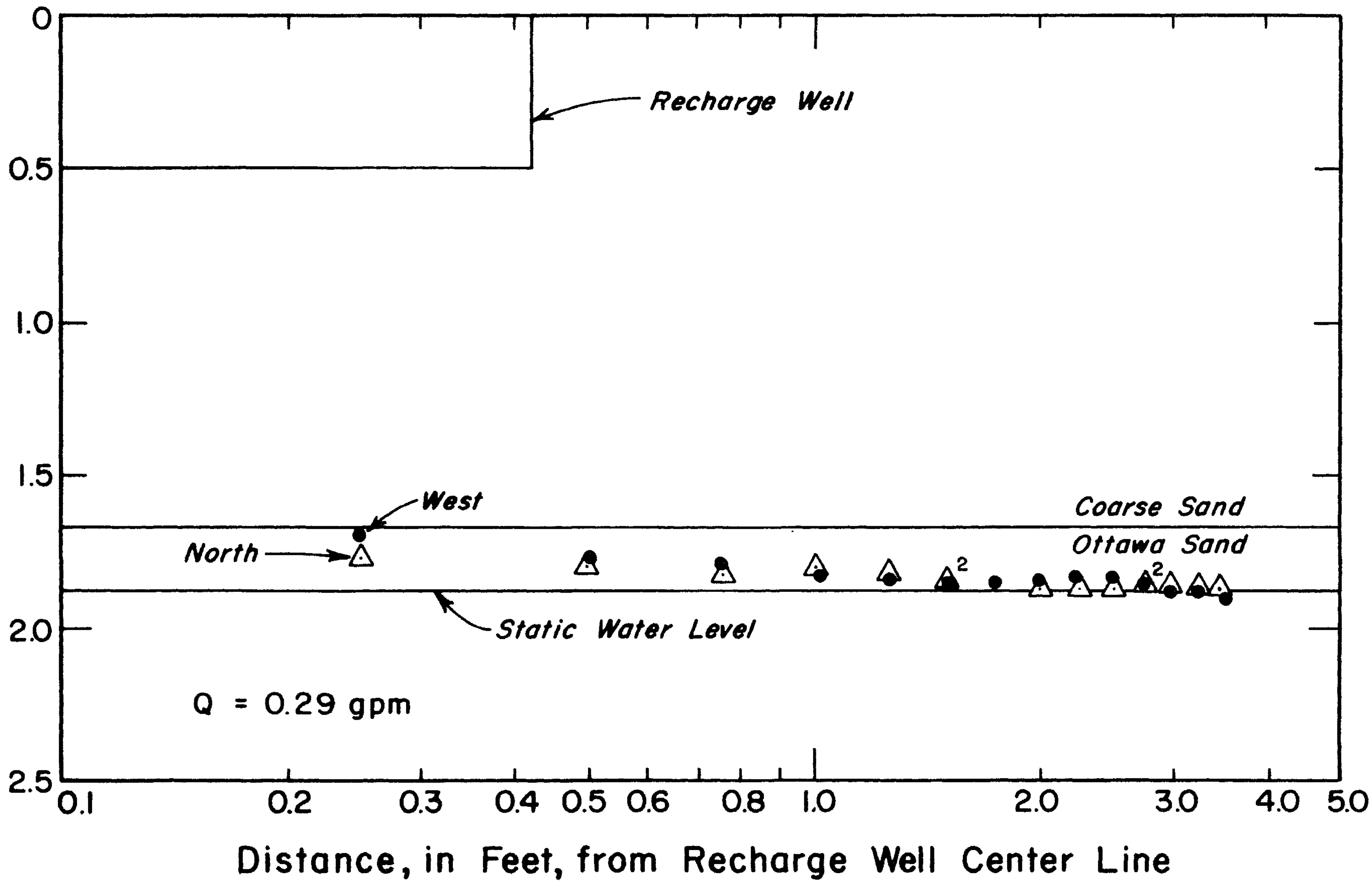

Figure 37. Shape of cone of recharge where Q = 0.29 gpm in a two-layered sequence.

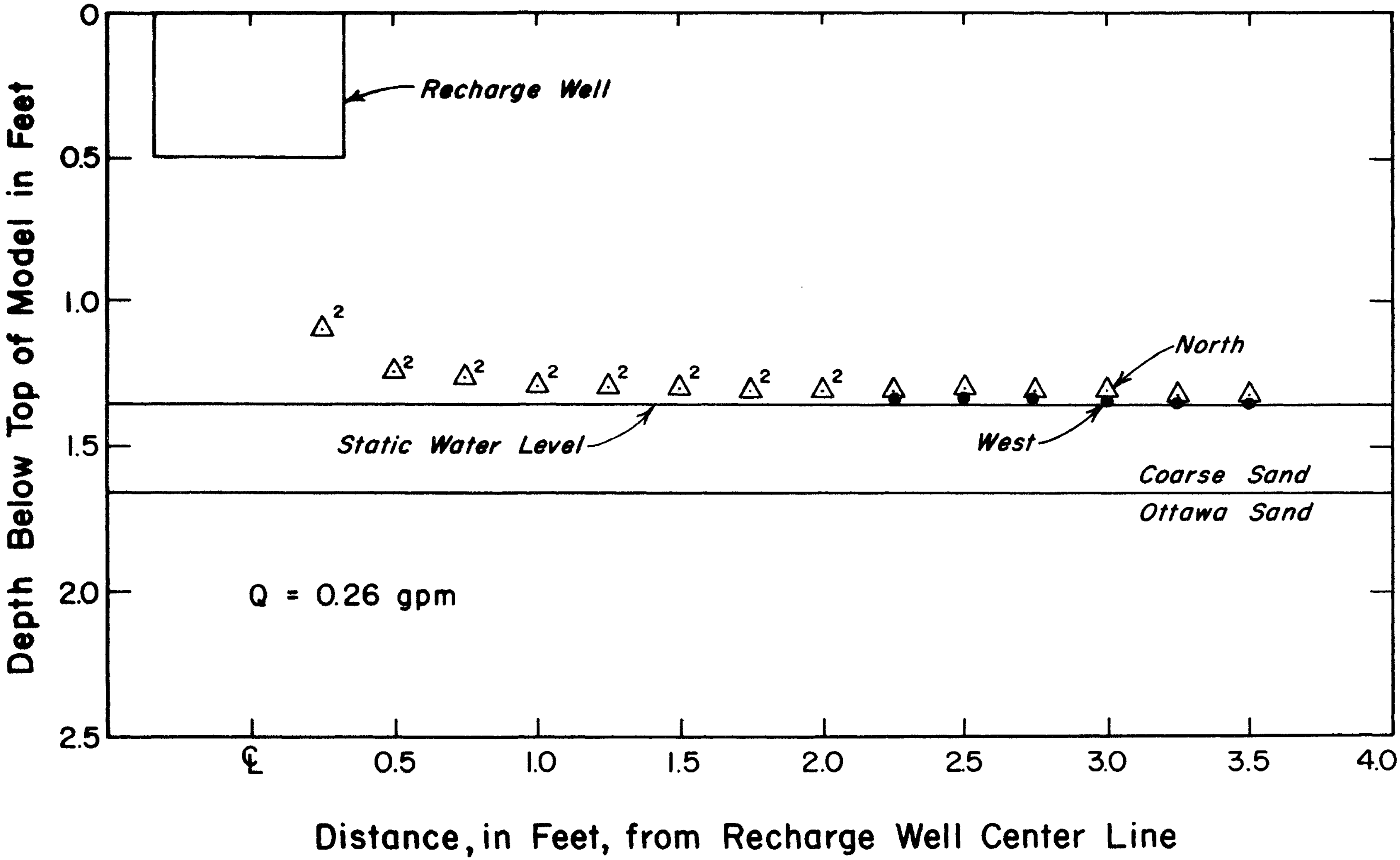

Figure 38. Shape of cone of recharge where Q = 0.26 gpm in a two-layered sequence, high static level.

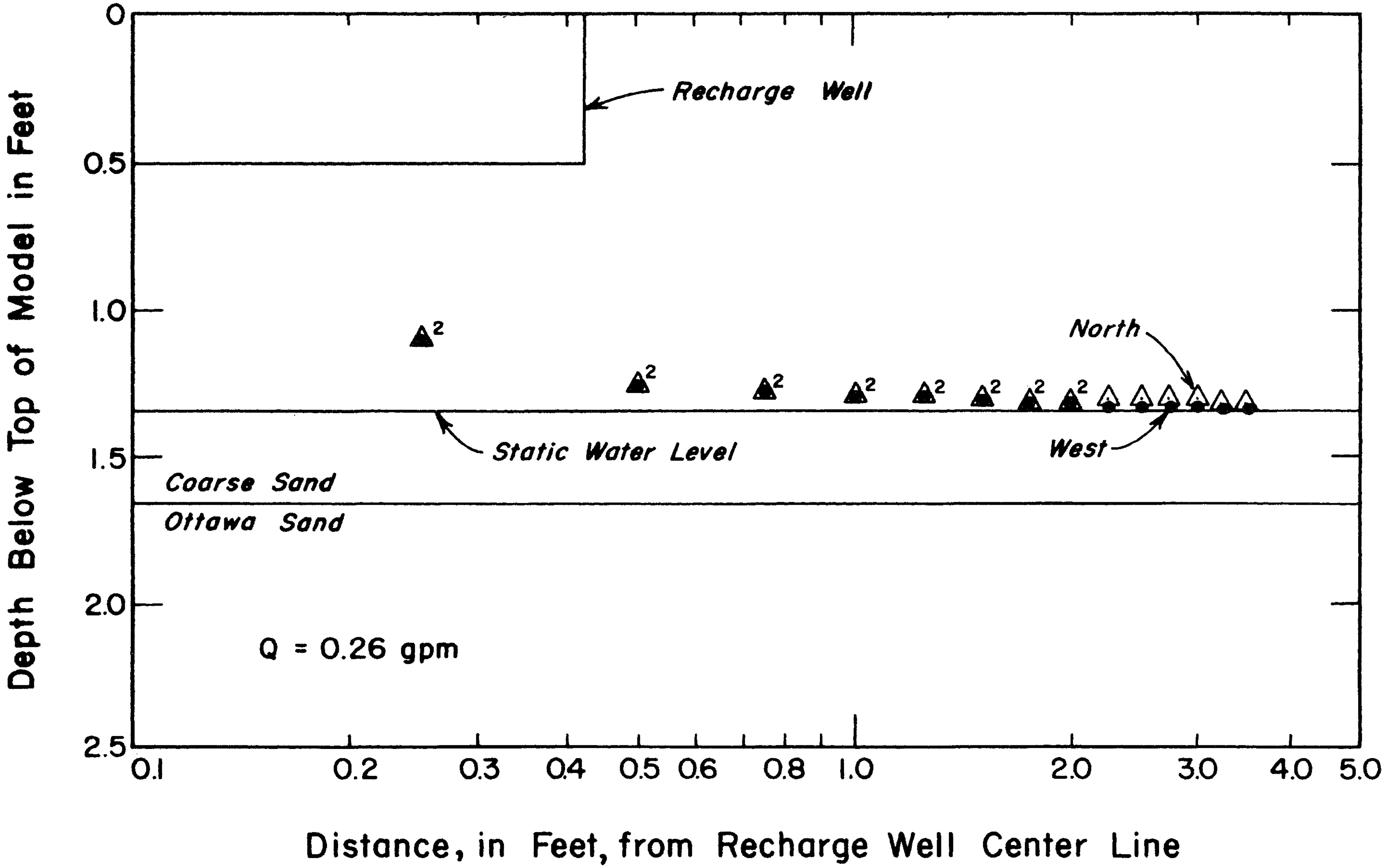

Figure 39. Shape of cone of recharge where Q = 0.26 gpm in a two-layered sequence, high static level.

measured (Table 6). Plots showing the shape in perpendicular directions of the cone of recharge are shown in Figures 40 and 41.

Test D. By maintaining a static water level at a depth of 1.77 feet or 0.11 feet below the coarse sand-Ottawa sand contact, it was possible to develop a cone of recharge that crossed the boundary. The rate of recharge was low, being only 0.256 gpm. at a water temperature of 23.5°C. The shape of the cone is distorted in the vicinity of the geologic boundary due to differences in permeability and grain size (Table 6). The shape of the cone of recharge in two directions as it existed under these conditions is shown in Figure 42 and 43.

Test E. The objective of this test was to examine the cone of recharge that would form at a relatively high recharge rate in such a manner that the water table would transect the geologic boundary. Deaired water was pumped into the well at a constant rate of 0.98 gpm. at temperatures of 22.1°C and 23.0°C. The static water level and the level maintained in the models end tank reservours was 1.73 feet. The depth to water was measured after the occurrence of steady-state conditions (Table 6). Graphic plots of the cone are shown in Figures 44 and 45.

Experiment 6.

During this experiment, five tests were conducted at various recharge rates using a three-layer sequence of clastic material. The objective was to examine the cone of recharge that would develop under these various sets of conditions. The model contained Ottawa sand (2.34 feet thick), overlain by 0.73 feet of coarse sand, which, in turn was covered by 0.92

Table 6. Data for Experiment 5, Tests A - E

<u>Test A</u> Stratified Model (two layers)

Q = 0.29 gpm T = 24°C		Q = 0.29 T = 25.1°C	
Distance, in feet, from recharge well centerline (west)	Depth, in feet, to water surface below top of model	Distance, in feet, from recharge well centerline (north)	Depth, in feet, to water surface below top of model
0.25	1.70	0.25	1.76
0.50	1.76	0.50	1.78
0.75	1.80	0.75	1.82
1.00	1.83	1.00	1.82
1.25	1.84	1.25	1.83
1.50	1.84	1.50	1.84
1.75	1.85	1.75	1.85
2.00	1.85	2.00	1.86
2.25	1.85	2.25	1.86
2.50	1.85	2.50	1.86
2.75	1.86	2.75	1.86
3.00	1.88	3.00	1.86
3.25	1.88	3.25	1.86
3.50	1.88	3.48	1.87
End tank	1.88	End tank	1.88

Test B   Stratified Model (two layers)

Q = 0.26 gpm   T = 21.0°C

Distance, in feet, from recharge well centerline (west)	Depth, in feet, to water surface below top of model
0.25	1.10
0.50	1.25
0.75	1.27
1.00	1.27
1.25	1.29
1.50	1.30
1.75	1.31
2.00	1.33
2.25	1.33
2.50	1.33
2.75	1.33
3.00	1.35
3.25	1.35
3.50	1.35
End Tank	1.35

Q = 0.26 gpm   T = 22.0°C

Distance, in feet, from recharge well centerline (north)	Depth in feet, to water surface below top of model
0.35	1.10
0.50	1.25
0.75	1.27
1.00	1.29
1.25	1.29
1.50	1.30
1.75	1.31
2.00	1.31
2.25	1.31
2.50	1.31
3.00	1.32
3.25	1.33
3.50	1.33
End tank	1.35

Test C   Stratified Model (two layers)

Q = 1.07 gpm T = 25°C		Q = 1.07 gpm T = 25°C	
Distance, in feet, from recharge well centerline (west)	Depth, in feet, to water surface below top of model	Distance, in feet, from recharge well centerline (north)	Depth, in feet, to water surface below top of model
0.25	0.99	0.25	0.91
0.50	1.03	0.50	0.98
0.75	1.06	0.75	1.06
1.00	1.14	1.00	1.10
1.25	1.17	1.25	1.13
1.50	1.22	1.50	1.20
1.75	1.22	1.75	1.21
2.00	1.26	2.00	1.25
2.25	1.26	2.25	1.26
2.50	1.30	2.50	1.27
2.75	1.30	2.75	1.27
3.00	1.30	3.00	1.29
3.25	1.31	3.25	1.29
3.50	1.32	3.50	1.29
End tank	1.35	3.75	1.29
		End tank	1.35

<u>Test D</u> Stratified Model (two layers)

Q = 0.256 gpm T = 23.5°C		Q = 1.07 gpm T = 25°C	
Distance, in feet, from recharge well centerline (west)	Depth, in feet, to water surface below top of model	Distance, in feet, from recharge well centerline (north)	Depth, in feet, to water surface below top of model
0.25	1.65	0.18	1.53
0.50	1.66	0.50	1.70
0.75	1.68	0.75	1.70
1.00	1.69	1.00	1.70
1.25	1.70	1.25	1.72
1.50	1.71	1.50	1.72
1.75	1.71	1.75	1.73
2.00	1.73	2.00	1.73
2.25	1.73	2.25	1.74
2.50	1.73	2.50	1.74
2.75	1.73	2.75	1.74
3.00	1.74	3.00	1.74
3.25	1.74	3.25	1.74
3.50	1.75	3.50	1.75
End tank	1.77	End tank	1.77

<u>Test E</u> Stratified Model (two layers)

Q = 0.98 gpm T = 22.1°C		Q = 0.98 gpm T = 23.0°C	
Distance, in feet, from recharge well centerline (west)	Depth, in feet, to water surface below top of model	Distance, in feet, from recharge well centerline (north)	Depth, in feet, to water surface below top of model
0.00	1.20	0.15	1.30
0.50	1.48	0.50	1.38
0.75	1.54	0.75	1.48
1.00	1.55	1.00	1.55
1.25	1.56	1.25	1.59
1.50	1.59	1.50	1.64
1.75	1.61	1.75	1.65
2.00	1.64	2.00	1.68
2.25	1.65	2.25	1.69
2.50	1.68	2.50	1.69
2.75	1.68	2.75	1.70
3.00	1.70	3.00	1.71
3.25	1.70	3.25	1.71
3.50	1.70	3.50	1.72
End tank	1.73	End tank	1.73

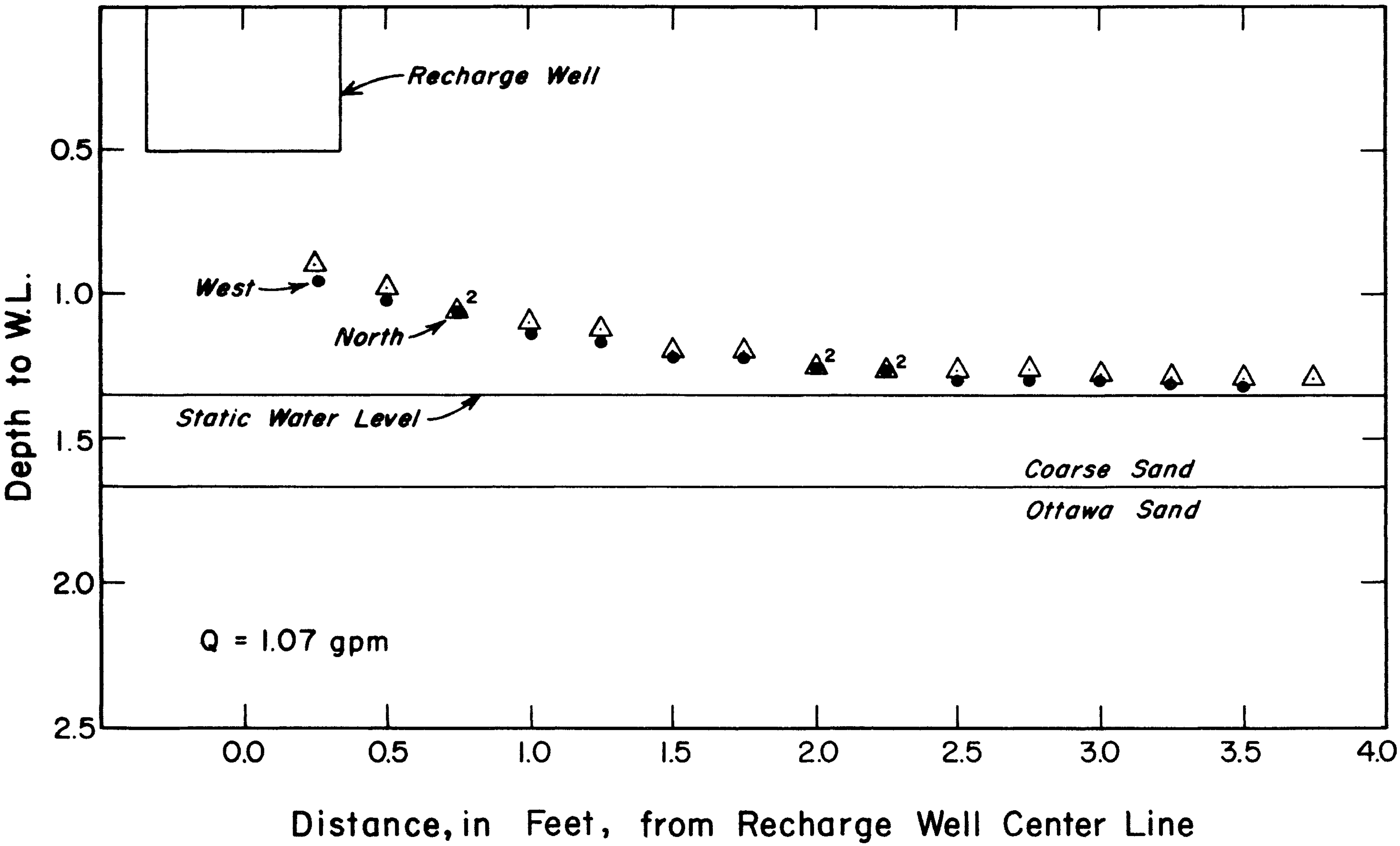

Figure 40. Shape of cone of recharge where Q = 1.07 gpm in a two-layered sequence, high static level.

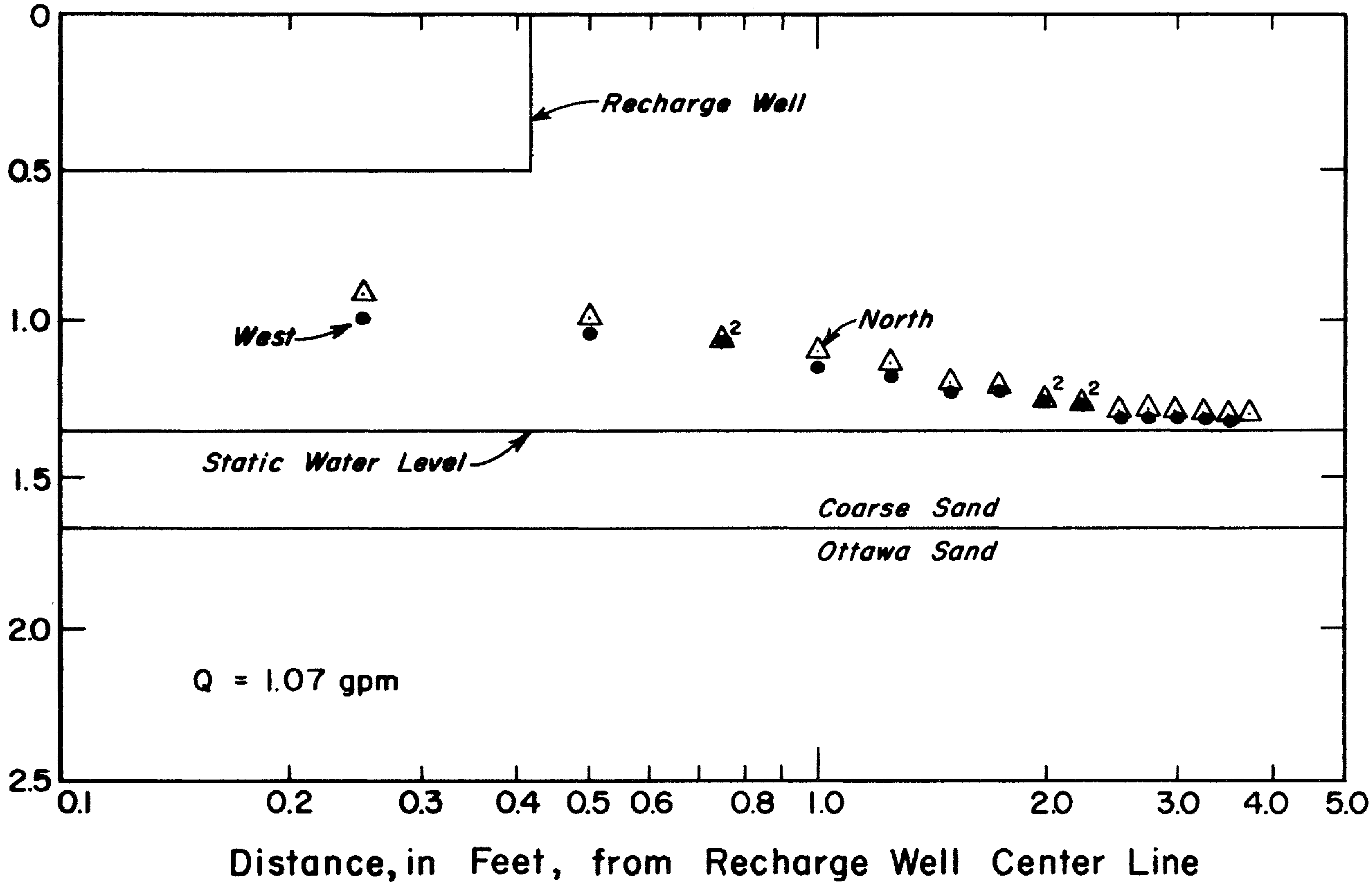

Figure 41. Shape of cone of recharge where Q = 1.07 gpm in a two-layered sequence, high static level.

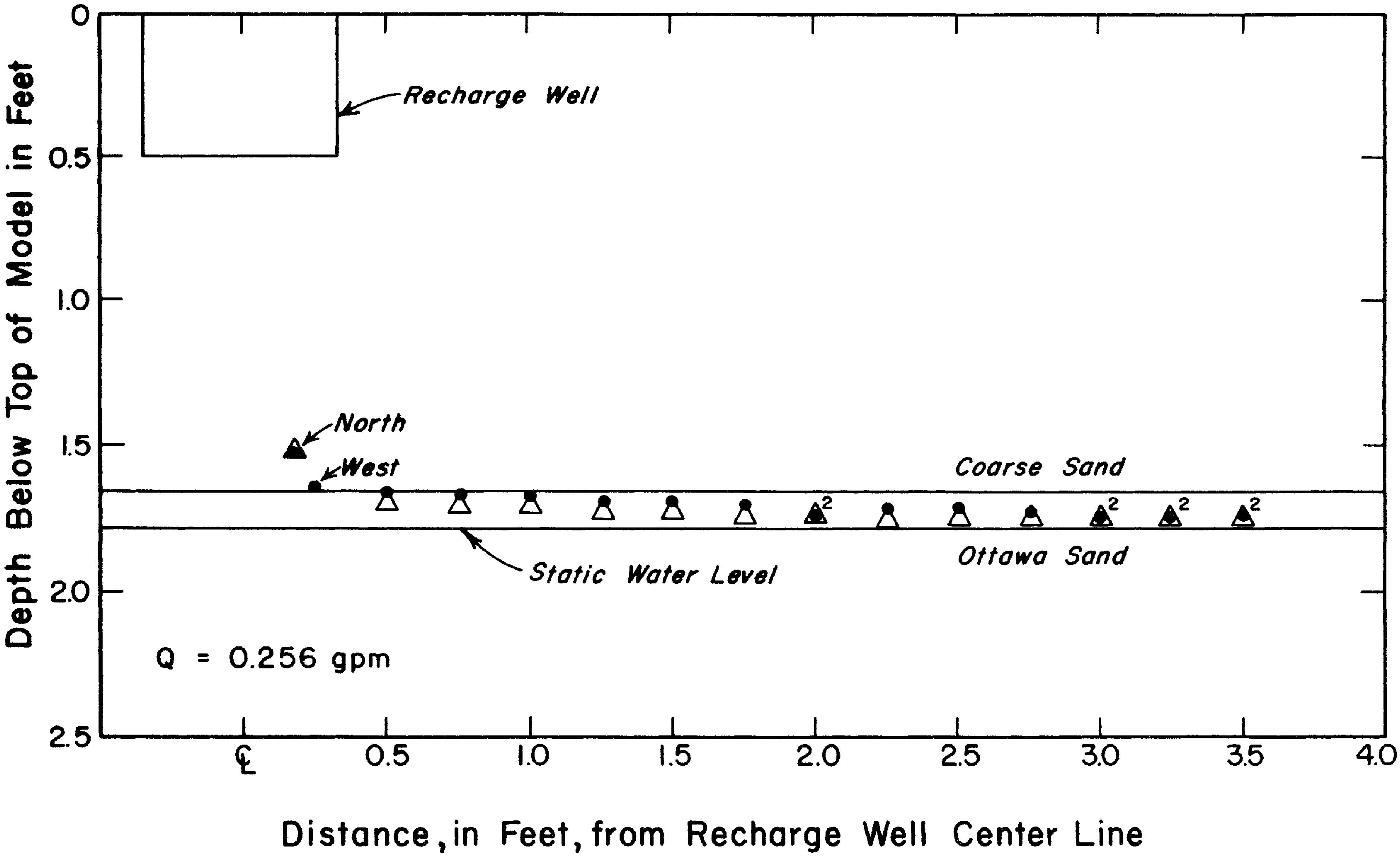

Figure 42. Shape of cone of recharge where Q = 0.256 gpm in a two-layered sequence.

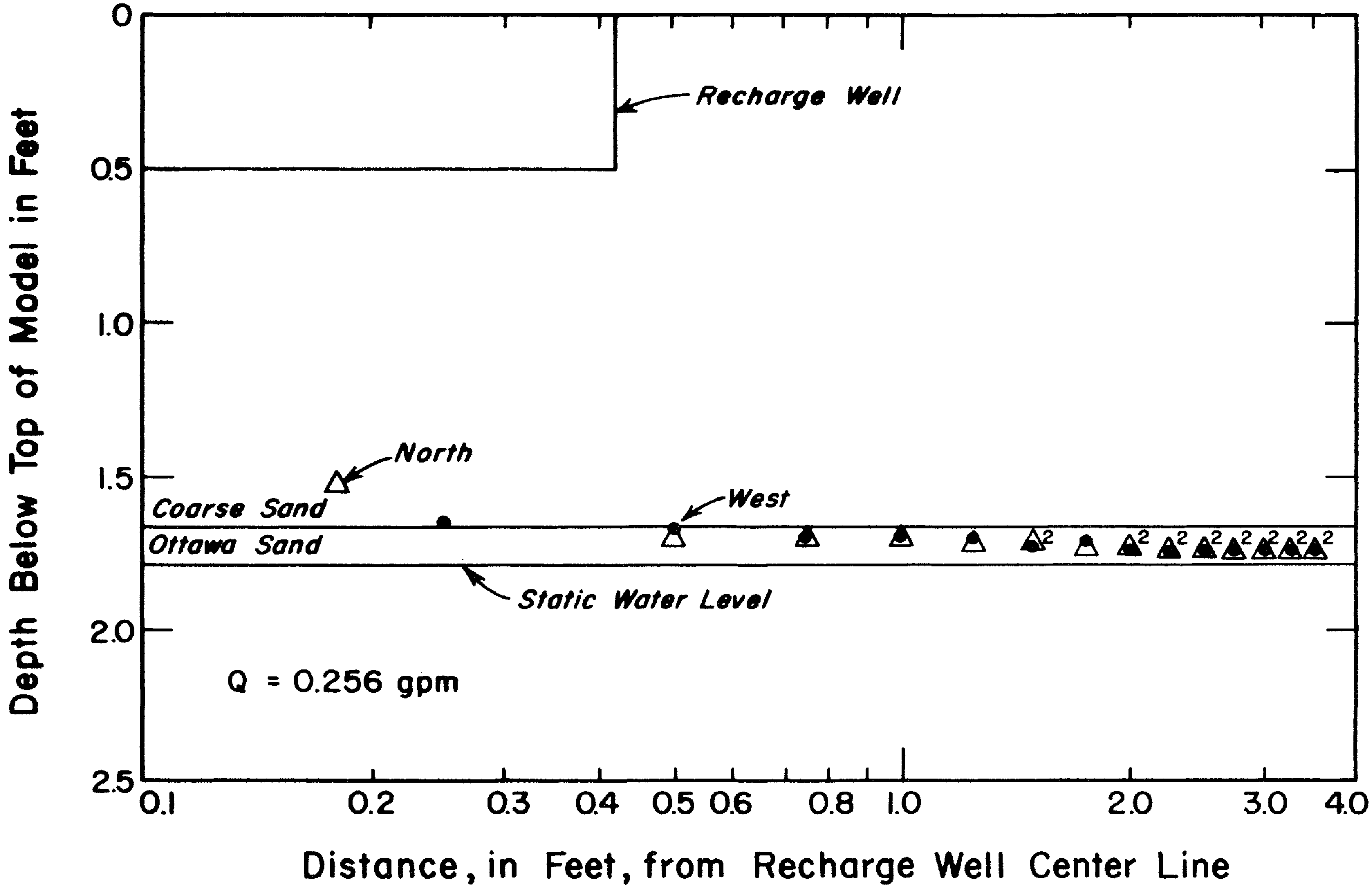

Figure 43. Shape of cone of recharge where Q = 0.256 gpm in a two-layered sequence.

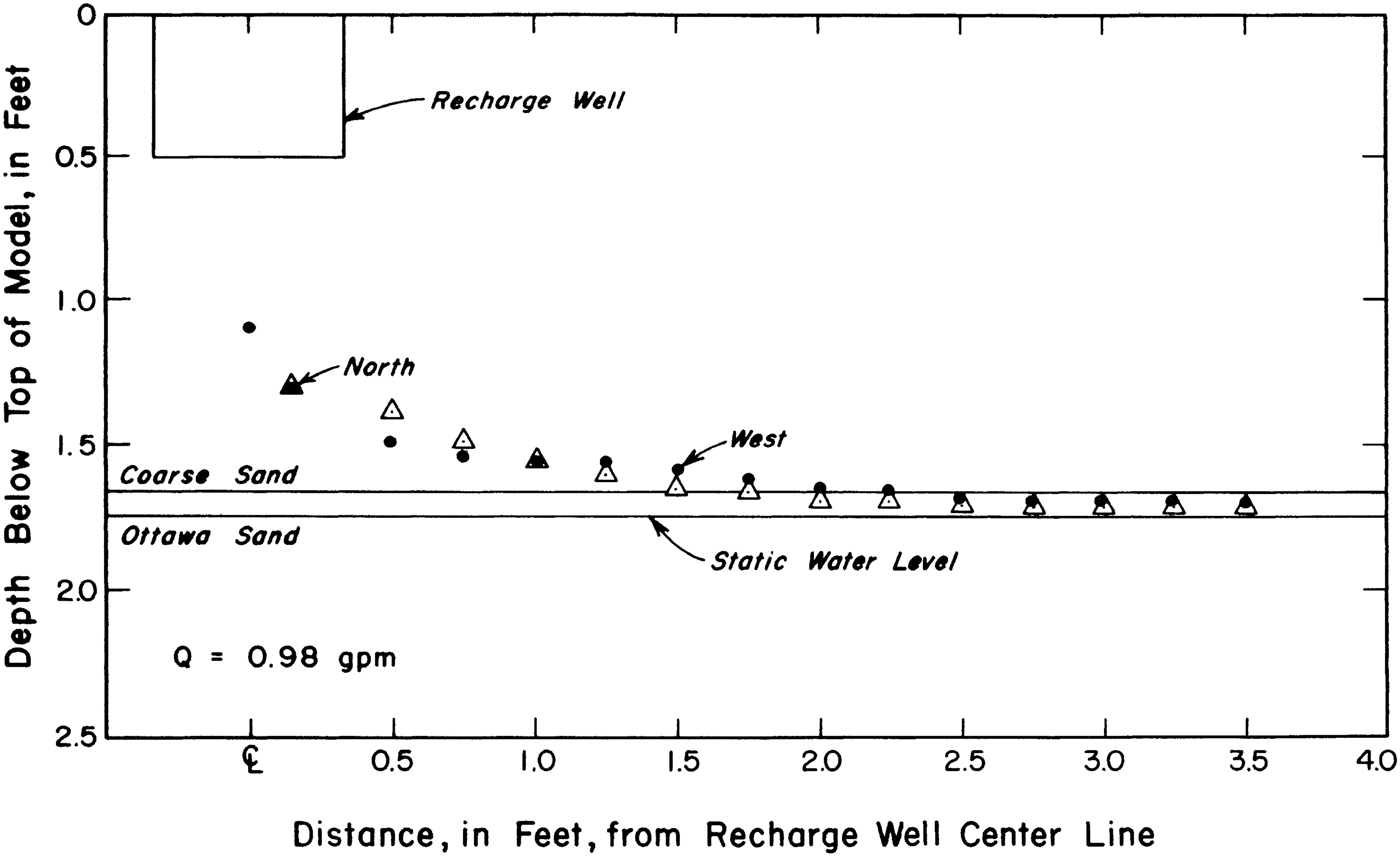

Figure 44. Shape of cone of recharge where Q = 0.98 gpm in a two-layered sequence.

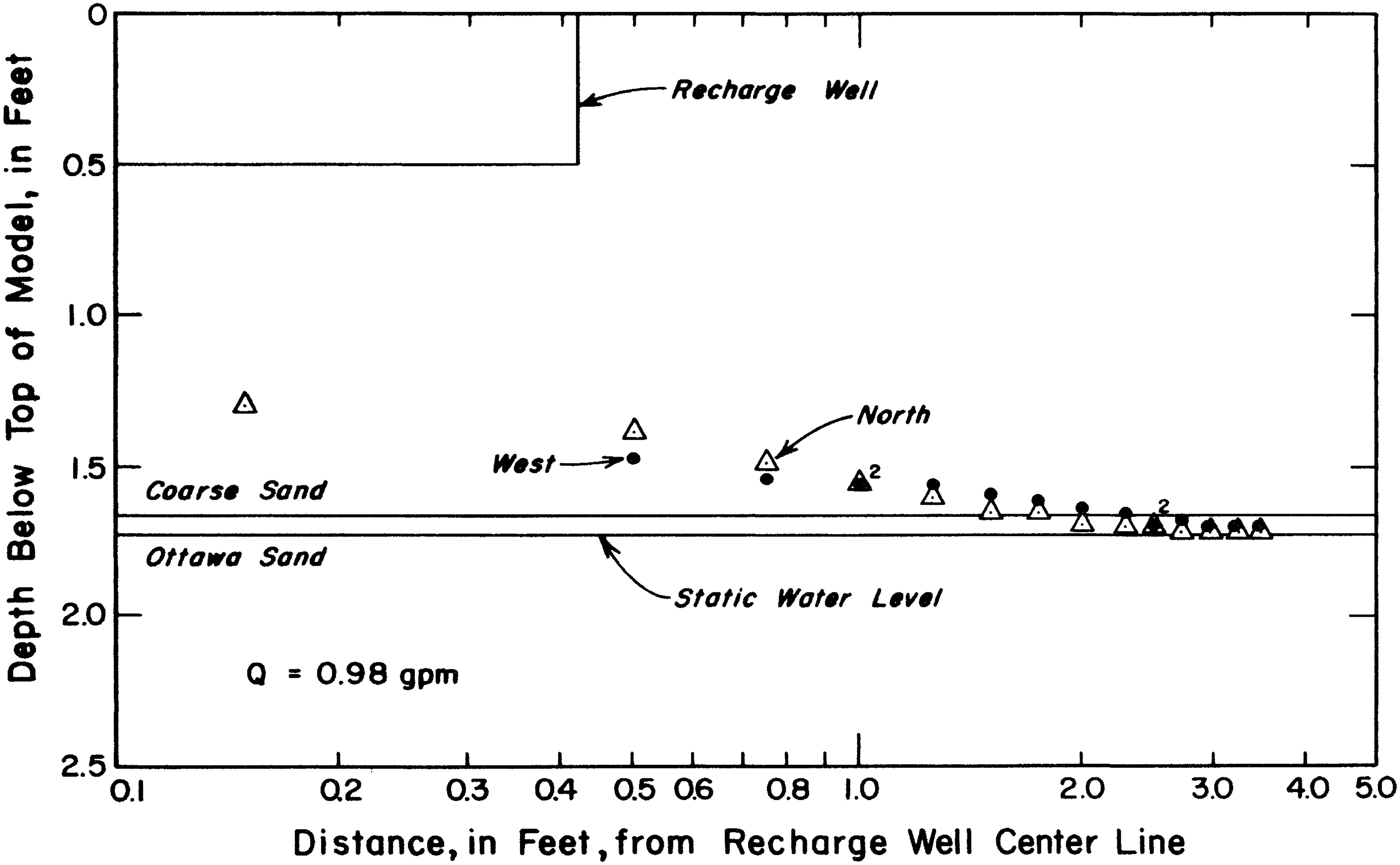

Figure 45. Shape of cone of recharge where Q = 0.98 gpm in a two-layered sequence.

feet of Ottawa sand. The recharge well was inserted into the upper layer to a depth of 0.36 feet (Fig. 46). The static water level ranged from a low of 1.88 feet during tests A and B, to a high of 1.86 feet during test E. The rate of recharge was also changed from one test to another.

Test A. Static water level maintained in the lowest unit. Water was injected into the model at a rate of 1.1 gpm and the cone of recharge was measured in both north and west directions.

Test B. The static water level was maintained in the lowest unit. Water was injected into the model at a low recharge rate (0.251 gpm) and the cone of recharge was measured in directions perpendicular to the recharge well.

Test C. The static water level was established in the middle unit (coarse sand). Water was injected into the well at a high rate of 1.05 gpm. The resulting cone was determined in north and west directions.

Test D. The static water level was maintained in the middle unit and water was recharged at a low rate of 0.264 gpm. The position of the water table after it reached steady-shape conditions was measured in both the north and south directions.

Test E. The static water level was established in the upper Ottawa sand unit. Water was injected at a high

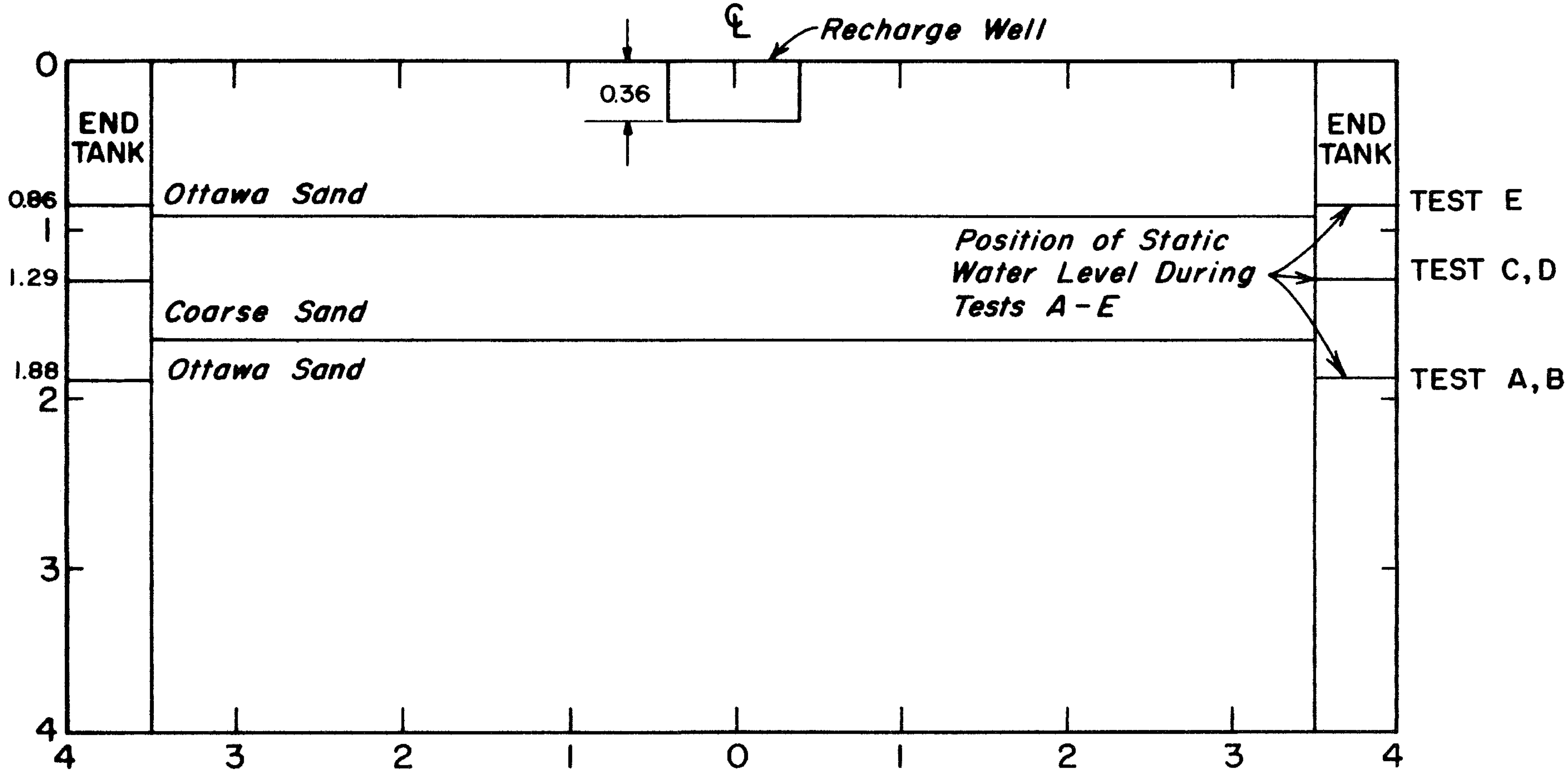

Figure 46. Model set up for experiment 6, tests 1-5.

rate (1.035 gpm) and then at a low rate (0.264 gpm). The cone of recharge was measured, in both cases, in only one direction (west).

Data representing these tests and graphs showing the shape of the water table are shown in Table 7 and Figures 47-56. The data indicate that a stratified reservoir tends to distort the shape of the cone of recharge in areas of the actual boundaries. In these experiments, changes in gradient are readily evident although not greatly pronounced largely because of the relatively samll difference in permeability.

All of the model experiments tend to indicate that the shape of the cone of recharge can be roughly calculated in field situations using readily available formulas. On the other hand, in highly stratified field situations and particularly where the layers have considerably different values of permeability and transmissibility, the cone may be so distorted that only crude approximation may be made. Additionally in highly permeable strata the cone of recharge may be largely confined to a small area within the near vicinity of a recharge well. Consequently, if the field situation is to be observed, observation wells will need to be placed close to the recharge well.

Table 7. Data for Experiment 6, Tests A - E

Test A Stratified Model (three layers)

Q = 1.1 gpm T = 22.5°C		Q = 1.04 gpm T = 23.5°C	
Distance, in feet, from recharge well centerline (west)	Depth, in feet, to water surface below top of model	Distance, in feet, from recharge well centerline (north)	Depth, in feet, to water surface below top of model
0.00	1.39	1.50	1.48
1.50	1.51	0.75	1.54
0.75	1.55	1.00	1.65
1.00	1.60	1.25	1.70
1.25	1.67	1.50	1.73
1.50	1.70	1.75	1.75
1.75	1.73	2.00	1.79
2.00	1.76	2.50	1.80
2.25	1.76	3.00	1.80
2.50	1.80	3.50	1.80
2.75	1.81	End tand	1.88
3.00	1.81		
3.25	1.82		
3.50	1.83		
End tank	1.88		

Test B Stratified Model (three layers)

Q = 0.256 gpm T = 23.5°C

Distance, in feet, from recharge well centerline (west)	Depth, in feet, to water surface below top of model
0.10	1.76
0.50	1.79
0.75	1.79
1.00	1.81
1.25	1.81
1.50	1.83
1.75	1.83
2.00	1.84
2.50	1.85
3.00	1.86
3.50	1.86
End tank	1.88

Q = 0.251 gpm T = 24°C

Distance, in feet, from recharge well centerline (north)	Depth, in feet, to water surface below top of model
0.50	1.77
0.75	1.80
1.00	1.82
1.25	1.84
1.50	1.85
2.00	1.85
2.50	1.86
3.00	1.86
3.50	1.86
End tank	1.88

Test C   Stratified Model (three layers)

Q = 1.05 gpm   T = 23.0°C

Distance, in feet, from recharge well centerline (west)	Depth, in feet, to water surface below top of model
0.30	0.85
0.50	0.88
0.75	0.98
1.00	1.06
1.25	1.12
1.50	1.16
1.75	1.23
2.00	1.23
2.50	1.24
3.00	1.25
3.50	1.28
End tank	1.29

Q = 0.251 gpm   T = 24°C

Distance, in feet, from recharge well centerline (north)	Depth, in feet, to water surface below top of model
0.50	0.90
0.75	0.98
1.00	1.08
1.25	1.12
1.50	1.15
1.75	1.19
2.00	1.22
2.50	1.24
3.00	1.25
3.50	1.28
End tank	1.29

<u>Test D</u> Stratified Model (three layers)

Q = 0.264 gpm T = 24.0°C		Q = 0.264 gpm T = 24.0°C	
Distance, in feet, from recharge well centerline (west)	Depth, in feet, to water surface below top of model	Distance, in feet, from recharge well centerline (north)	Depth, in feet, to water surface below top of model
0.10	0.90	0.10	0.92
0.50	0.23	0.50	1.24
0.75	1.25	0.75	1.24
1.00	1.27	1.00	1.28
1.25	1.27	1.25	1.28
1.50	1.27	1.50	1.28
2.00	1.29	2.00	1.29
2.50	1.29	2.50	1.29
3.00	1.29	3.00	1.29
3.50	1.29	3.50	1.29
End tank	1.29	End tank	1.29

<u>Test E</u> Stratified Model (three layers)

Q = 1.035 gpm T = 24.0°C

Distance, in feet, from recharge well centerline (west)	Depth, in feet, to water surface below top of model	Distance, in feet, from recharge well centerline (west)	Depth, in feet, to water surface below top of model
0.35	0.40	0.50	0.74
0.50	0.56	0.75	0.77
0.75	0.62	1.00	0.81
1.00	0.69	1.25	0.84
1.25	0.72	1.50	0.84
1.50	0.76	2.00	0.85
2.00	0.i0	2.50	0.85
2.50	0.84	3.00	0.85
3.00	0.86	3.50	0.86
3.50	0.86	End tank	0.86
End tank	0.86		

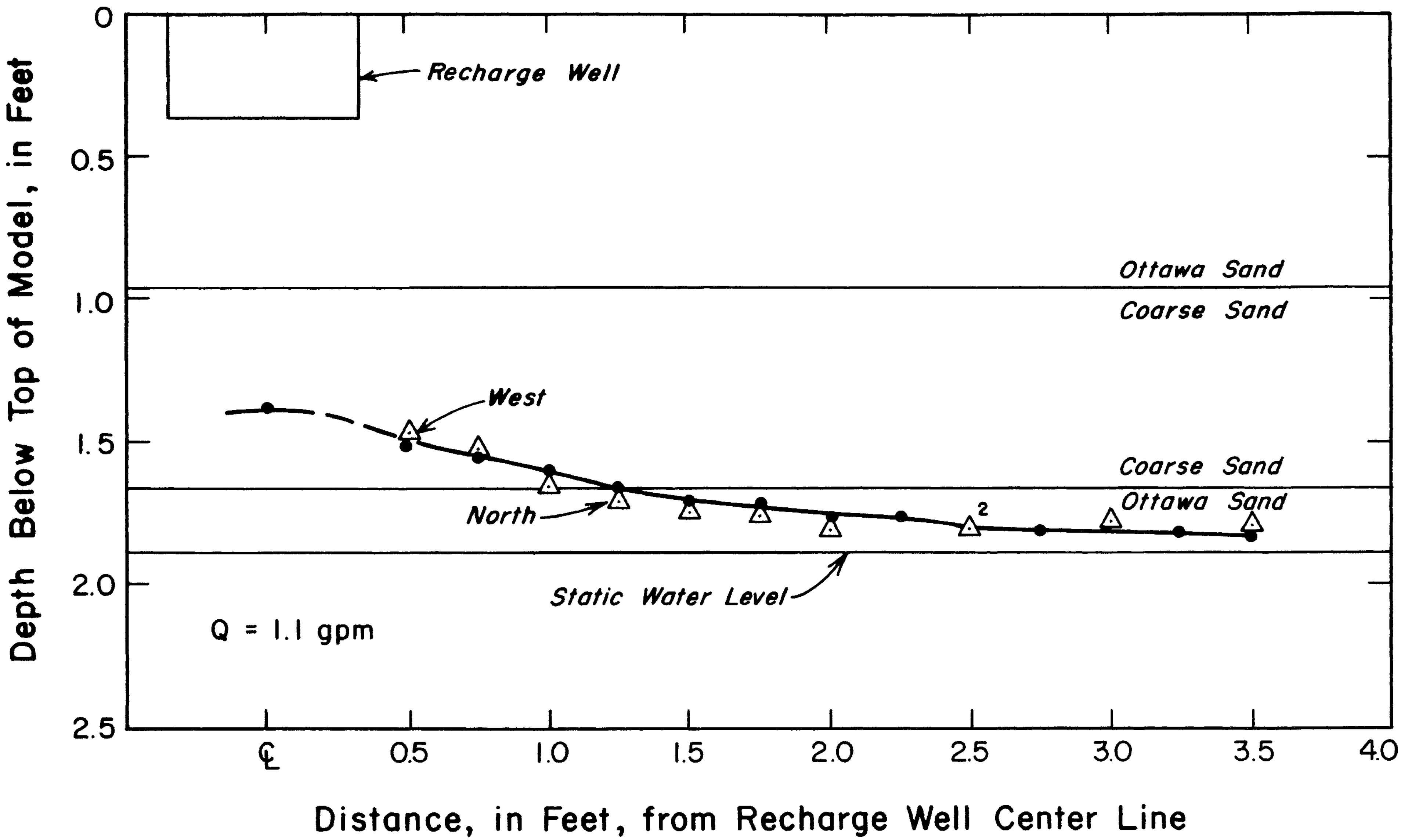

Figure 47. Shape of cone of recharge where Q = 1.1 gpm in a three-layered sequence.

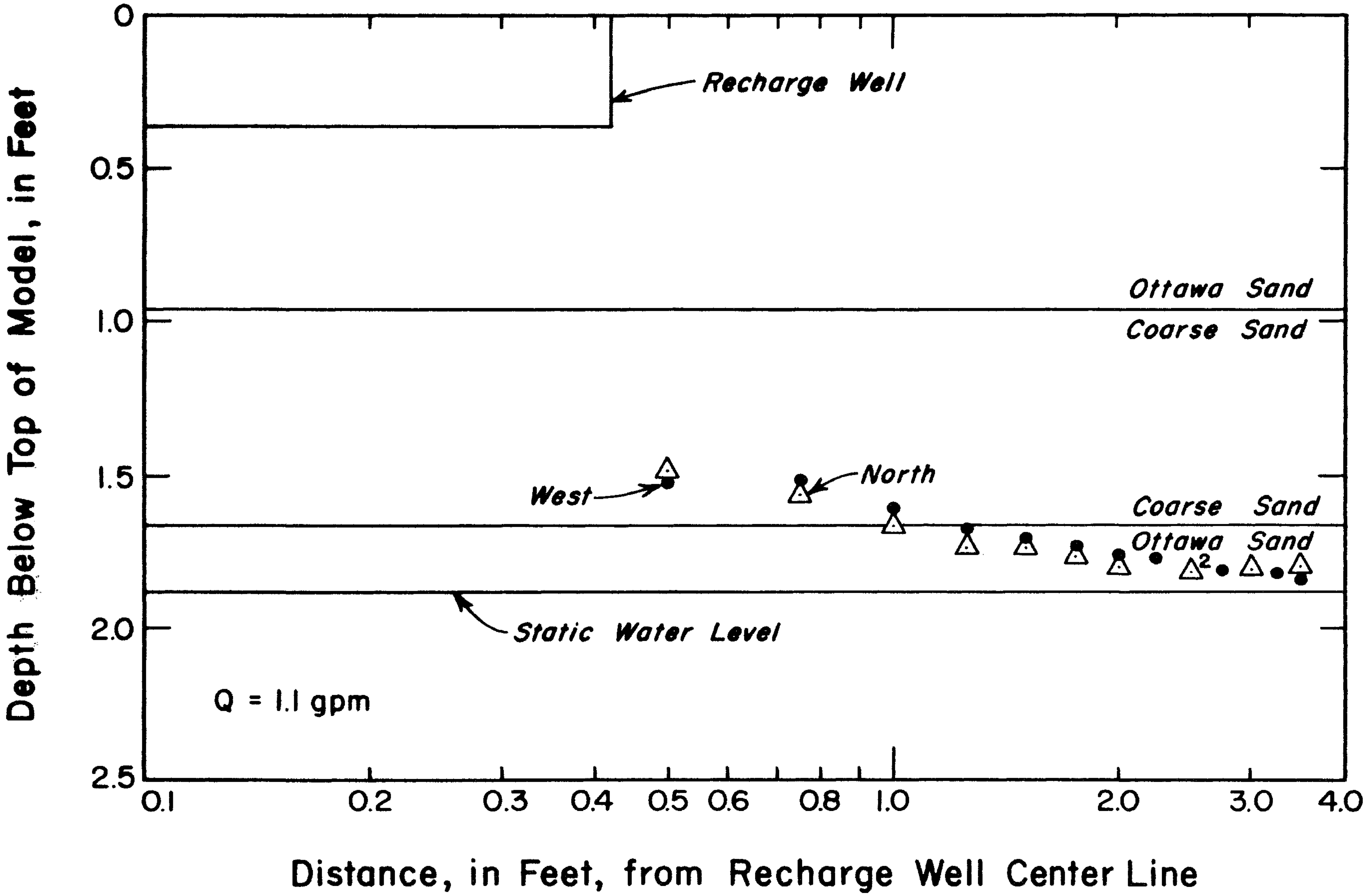

Figure 48. Shape of cone of recharge where Q = 1.1 gpm in a three-layered sequence.

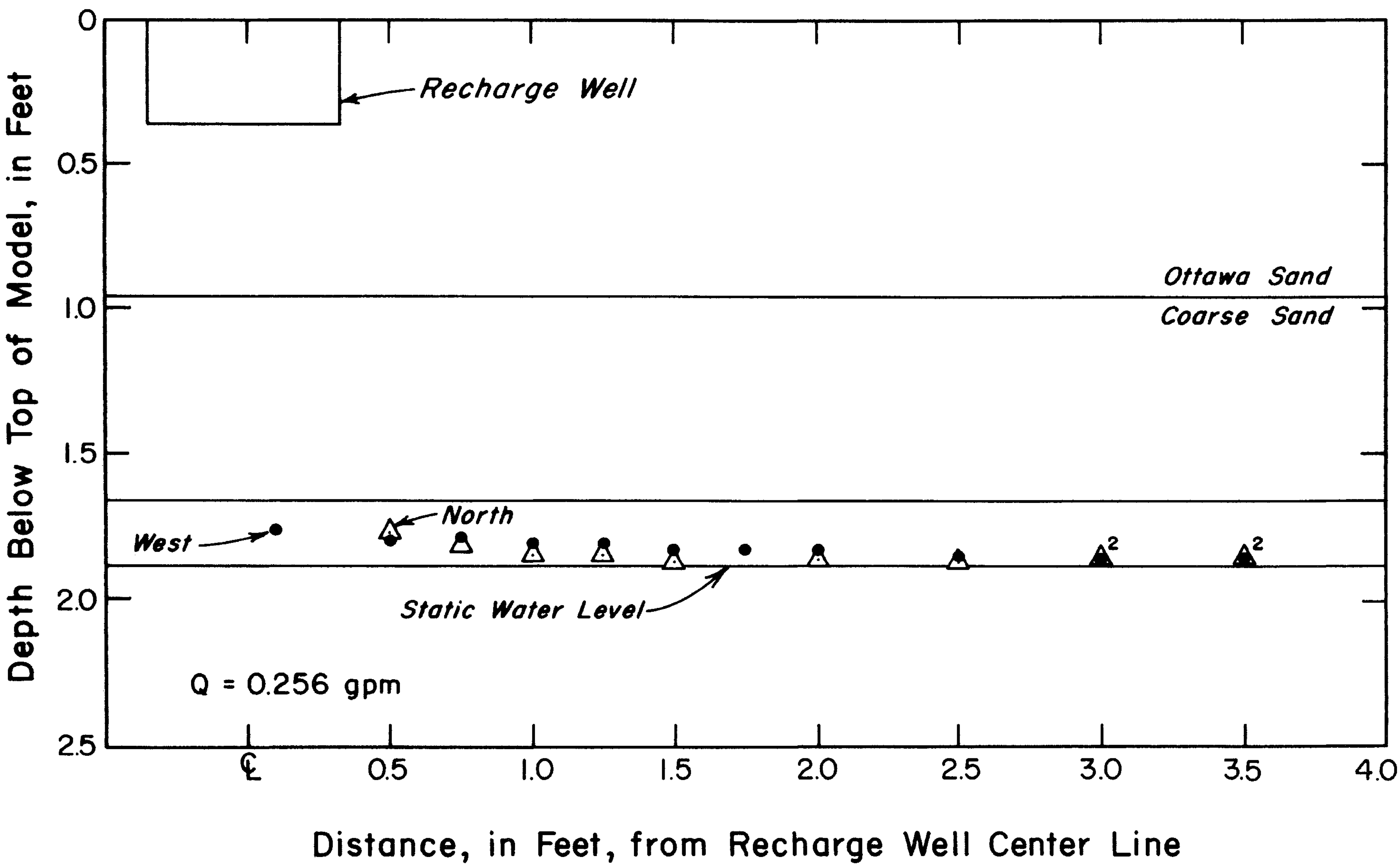

Figure 49. Shape of cone of recharge where Q = 0.256 gpm in a three-layered sequence.

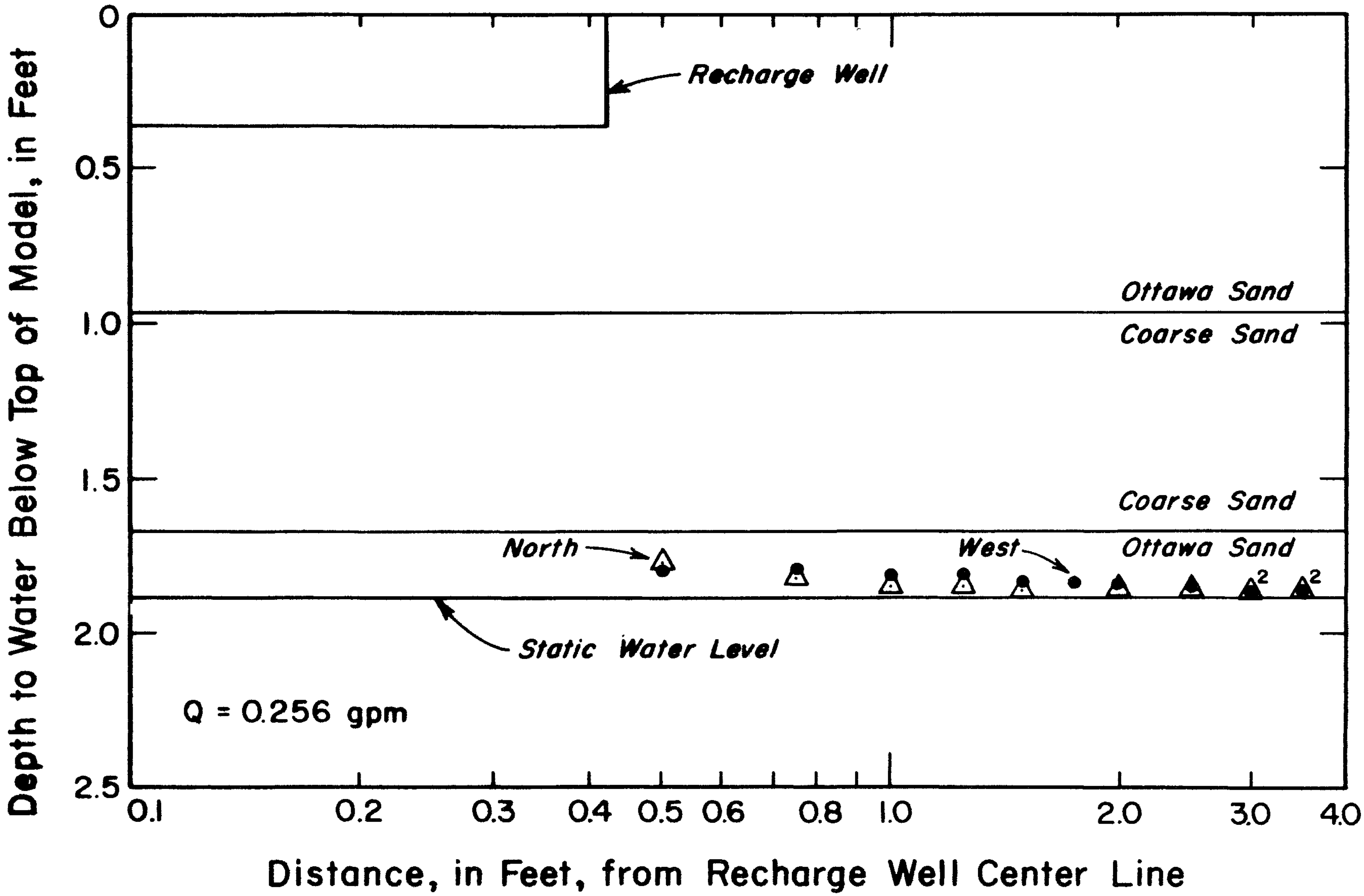

Figure 50. Shape of cone of recharge where Q = 0.256 gpm in a three-layered sequence.

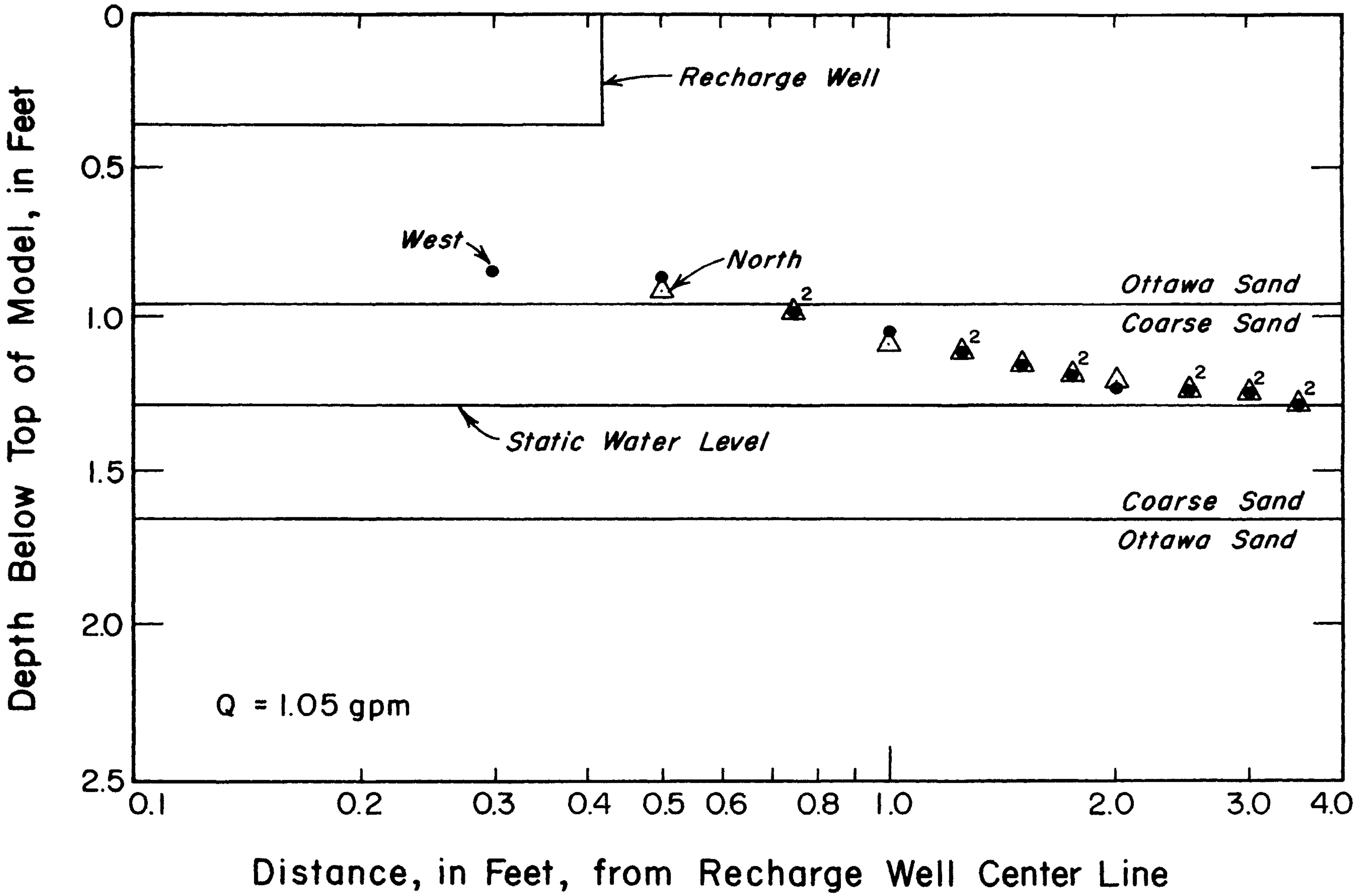

Figure 51. Shape of cone of recharge where Q = 1.05 gpm in a three-layered sequence.

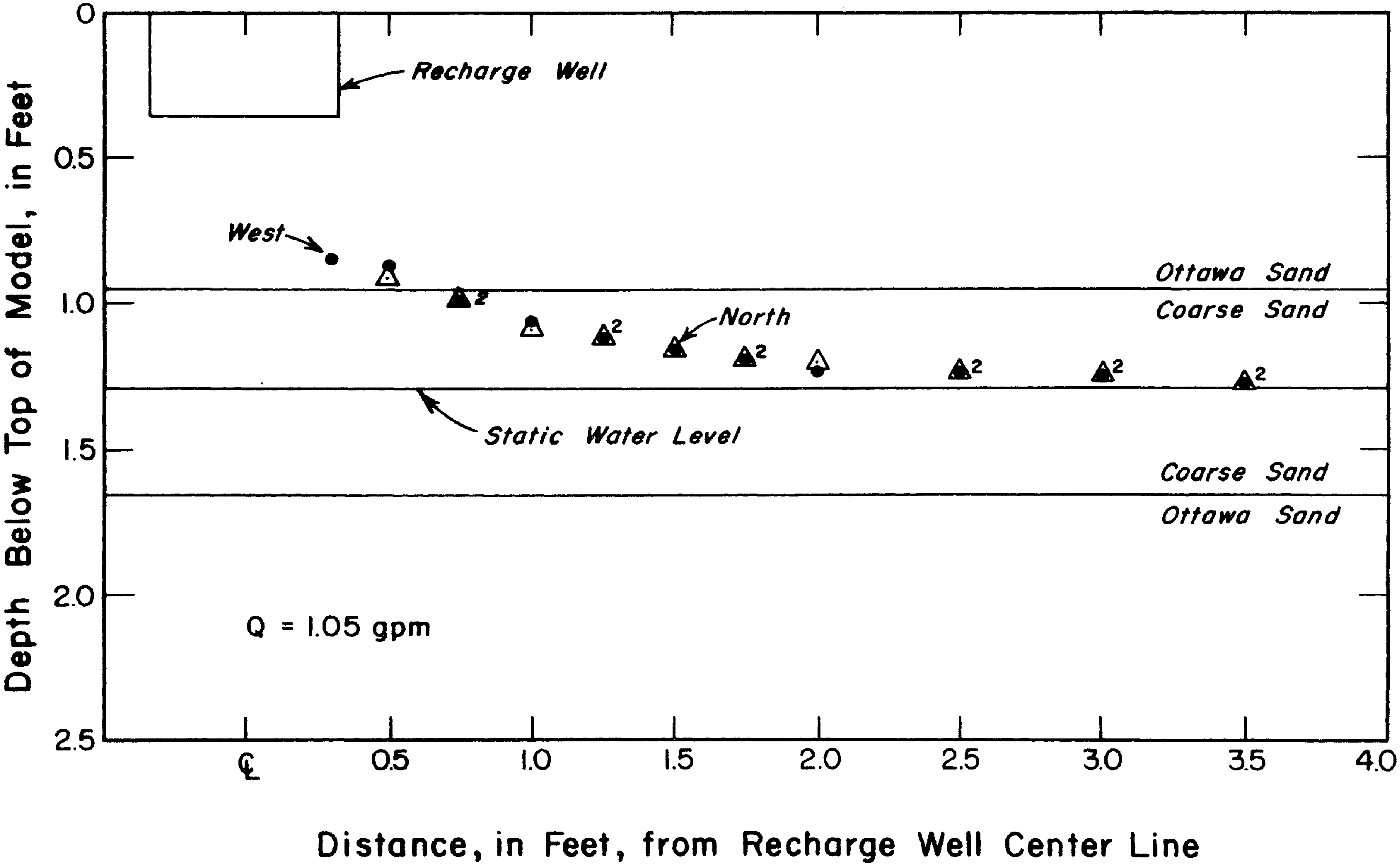

Figure 52. Shape of cone of recharge where Q = 1.05 gpm in a three-layered sequence.

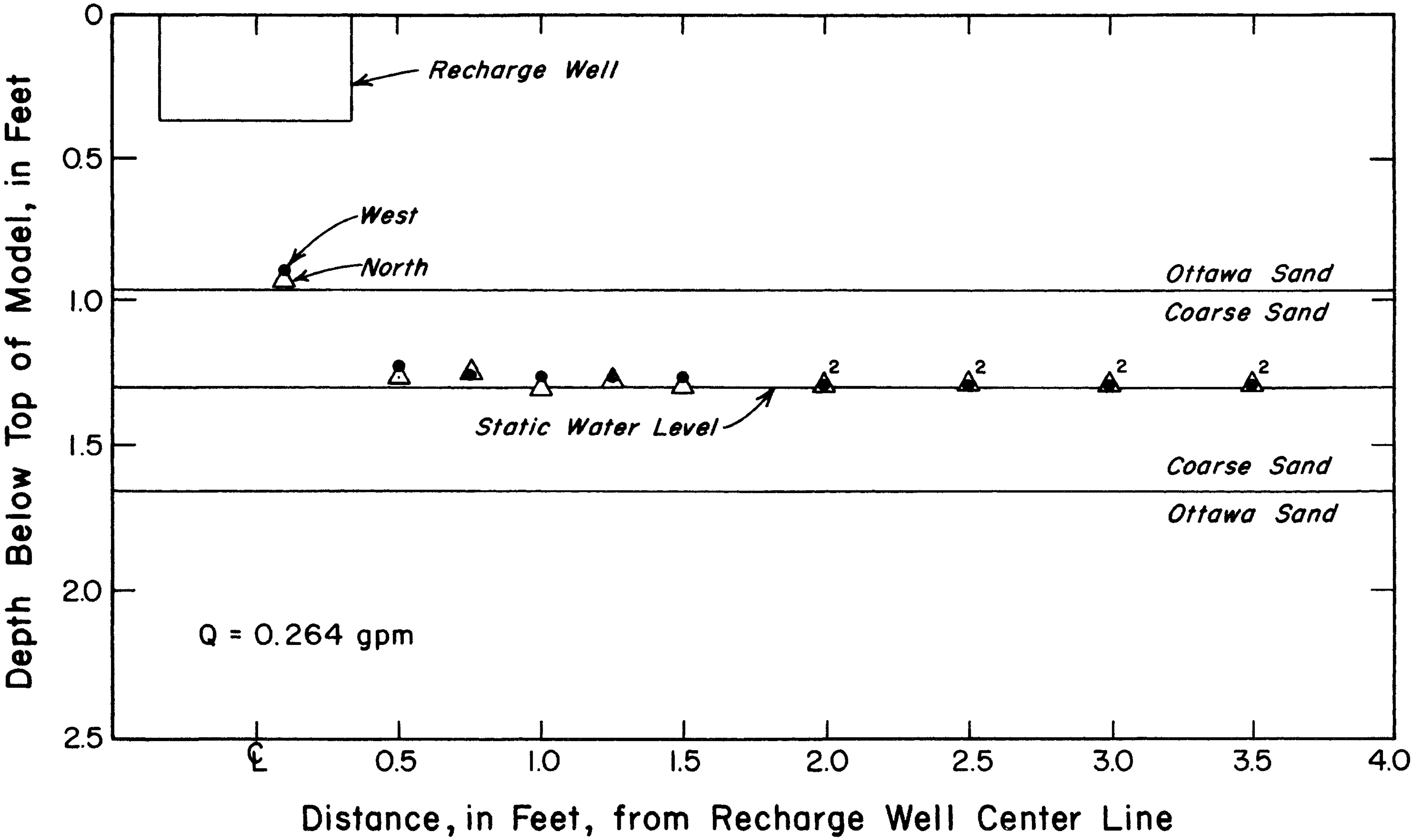

Figure 53. Shape of cone of recharge where Q = 0.264 gpm in a three-layered sequence.

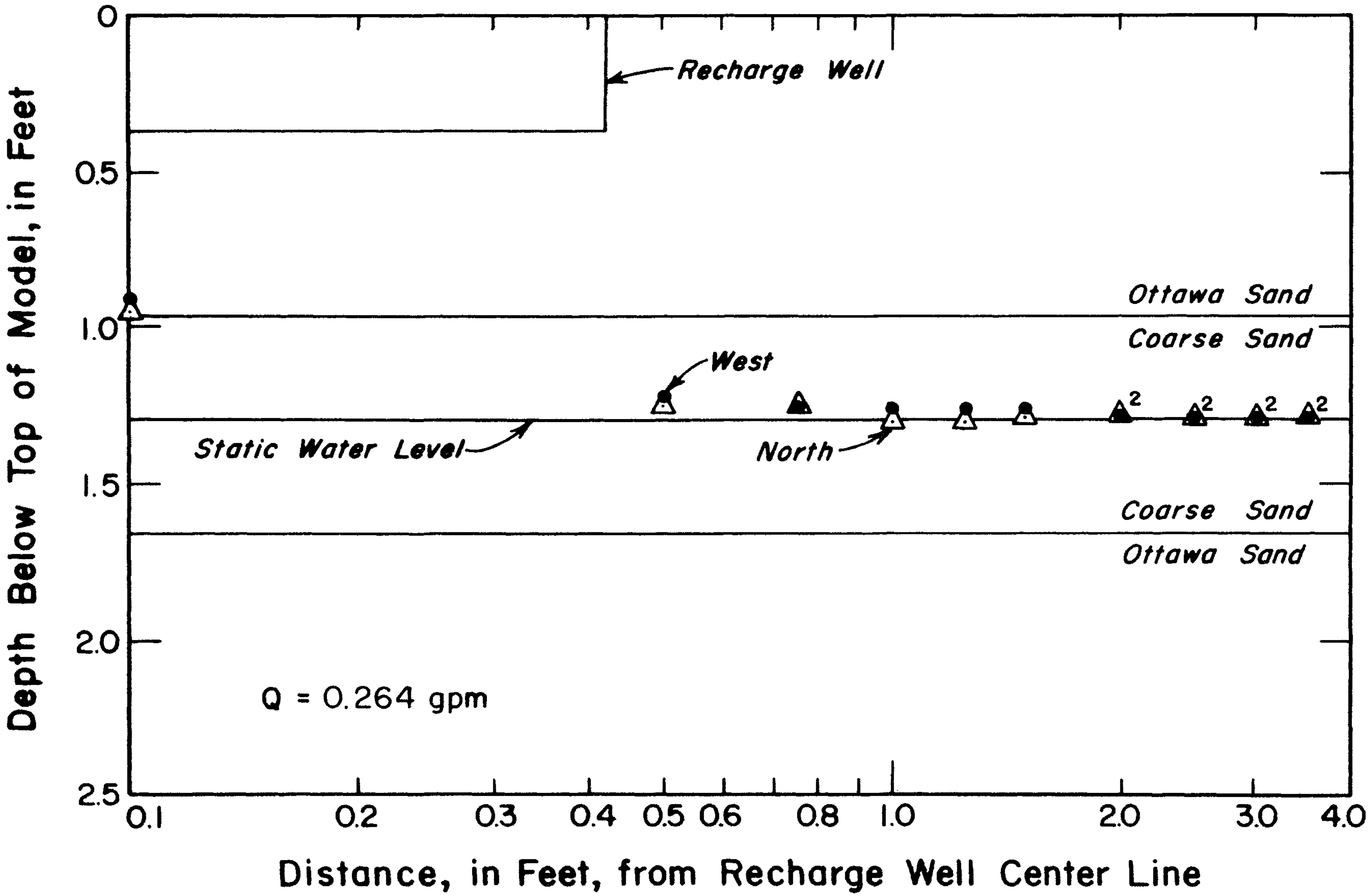

Figure 54. Shape of cone of recharge where Q = 0.264 gpm in a three-layered sequence.

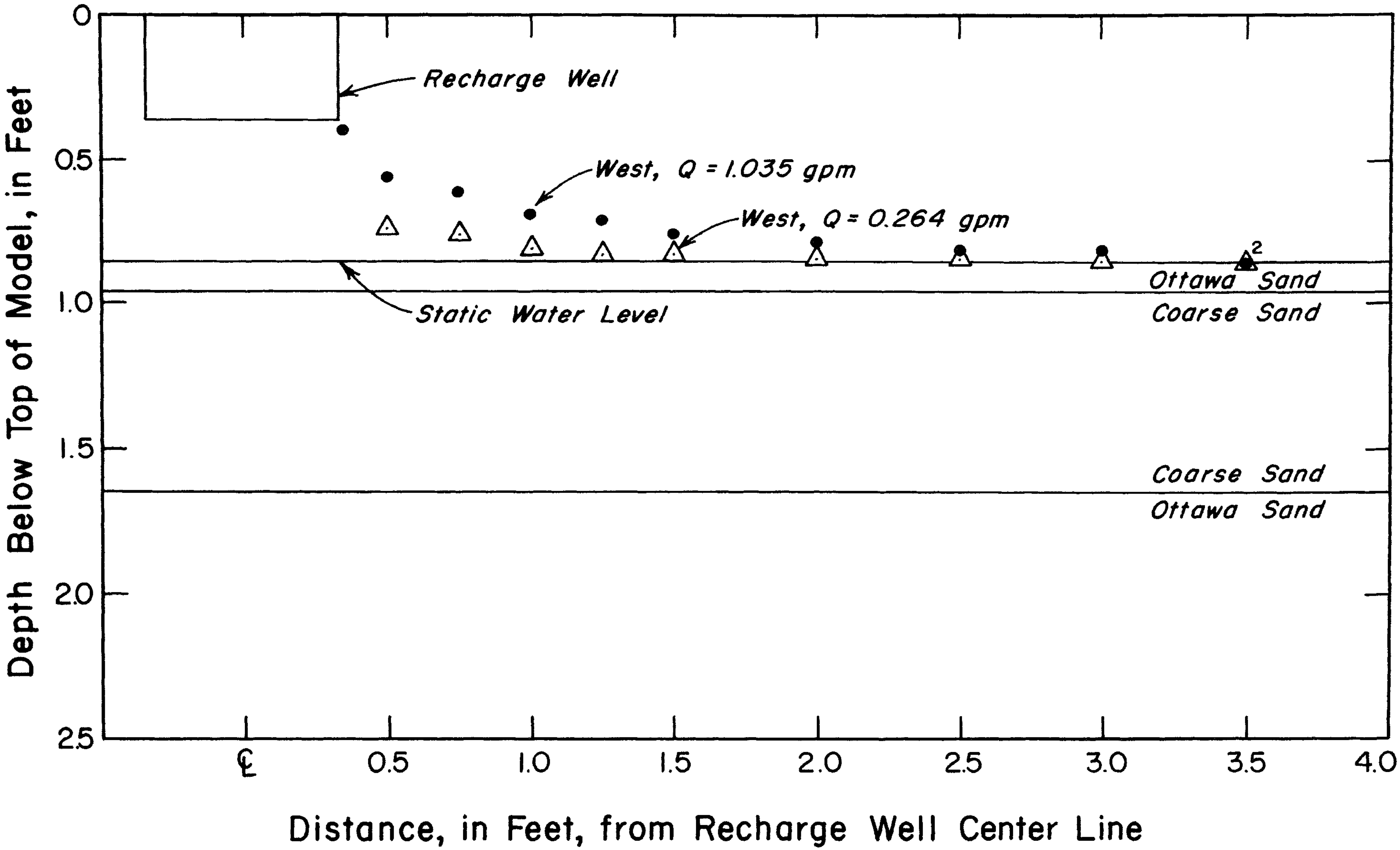

Figure 55. Shape of cone of recharge where Q = 0.264 and 1.035 gpm in a three-layered sequence.

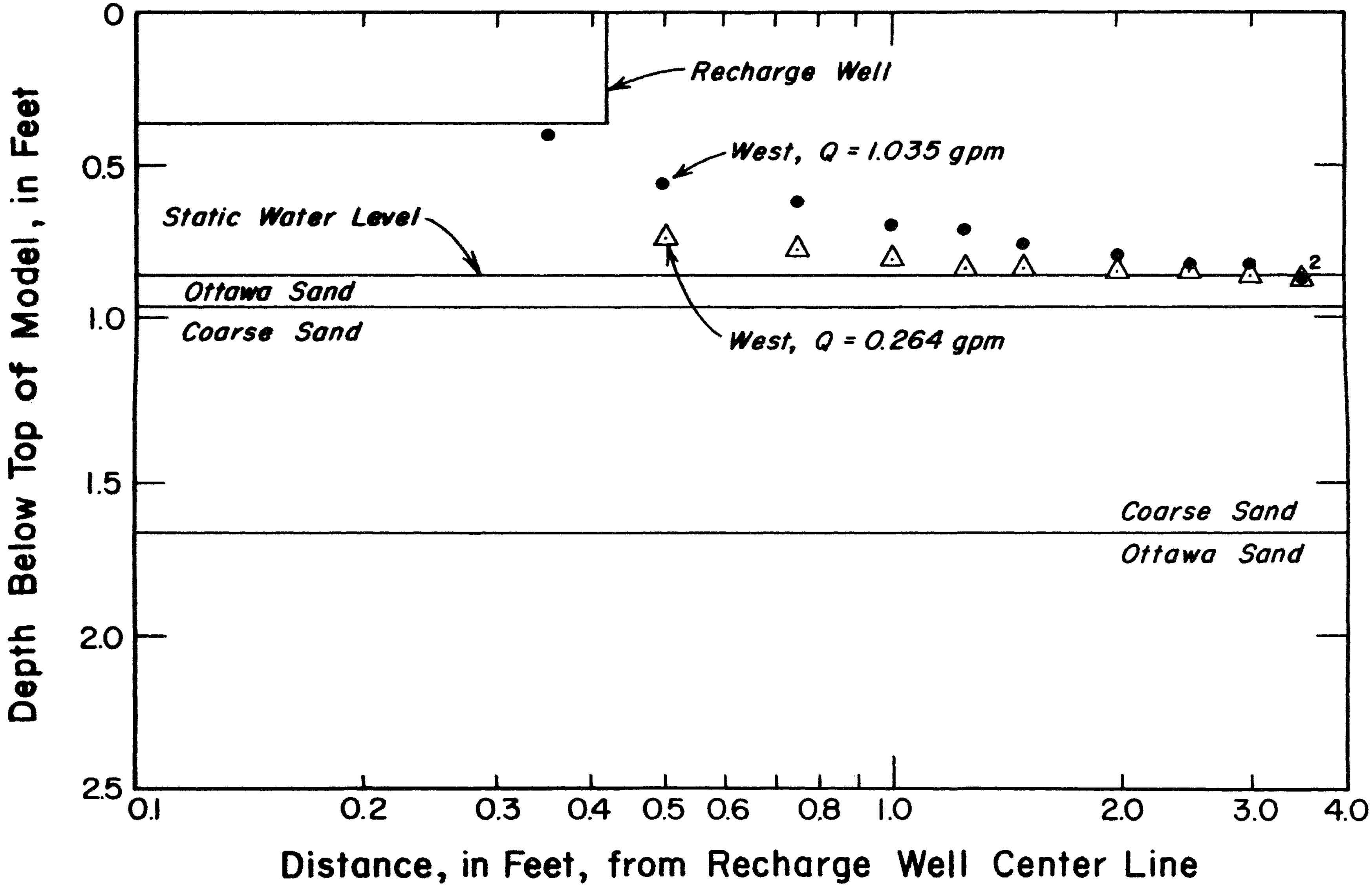

Figure 56. Shape of cone of recharge where Q = 0.264 and 1.035 gpm in a three-layered sequence.

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